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GREGORY P. TSCHEBOTARIOFF,[10] M.ASCE.—The great practical and theoretical value of studies of the type performed at Long Beach cannot be exaggerated, and all participants in this work and the sponsors of it deserve the highest commendation. A wealth of data has been obtained by such studies that could not be obtained by any other means. In the past there has been reluctance among practicing engineers to perform such measurements and, especially, to make their results public for fear of criticisms of their previous designs. The Board of Harbor Commissioners of the City of Long Beach is to be commended highly for taking a different attitude in this matter. The observations made should permit design improvements, not only at Long Beach, but in many other harbors, which otherwise could not have been accomplished. The author and his staff at the University of California are to be complimented highly for having performed this work, as is the Committee on Soil Mechanics of the Los Angeles Section, ASCE, for having sponsored it, and for having encouraged the publication and discussion of the results.

In order to facilitate the following discussion, four of the nineteen test stages reported in Fig. 12 by Mr. Duke have been selected as most significant. They are replotted at a larger scale in Fig. 18. The anchor forces are in kips per foot, and the bulkhead pressures are in kips per square foot.

The author has estimated that, during backfilling, the average lateral pressure coefficients varied from $K=0.7$ to $K=0.8$ (Fig. 13). This average value is, roughly, 60% higher than the value $K=0.44$, obtained by the writer under static conditions during a model test with silty sand at Princeton University, at Princeton, N. J. It is approximately 190% higher than the average value of 0.24 obtained during the several model tests with clean sand.[11] However, strong vibration was found, at Princeton, to increase the above static lateral pressure coefficients by 227% for silty sand and by from 58% to 111% for clean sand.[11]

Therefore, it is possible that the high K-values recorded by the author were caused by continued slight vibrations of the type produced by wave action, which caused the sand fill to "wedge itself in" behind the bulkhead while consolidating during the backfilling.

However, another explanation is also possible. It will be noted from Fig. 18, stage 1, that the pressure distribution was not entirely hydrostatic—the K-values increasing from $K=0.4$ above the anchor level to almost $K=1.00$ near the top of the rock dike. The latter high values may have been caused by "true arching" in the sand in a horizontal direction between the rock dike and the closely spaced batter piles of the anchorage, as shown by the arrows (II) in Fig. 19. This illustration is from a previous work by the writer.[12] No such batter piles were present during the Princeton model tests.

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[10] Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

[11] "Final Report. Large Scale Earth Pressure Tests with Model Flexible Bulkheads," by Gregory P. Tschebotarioff, submitted to Bureau of Yards and Docks, Dept. of the Navy, Princeton Univ., Princeton, N. J., January, 1949.

[12] "Einfluss der Gewölbebildung auf die Erddruckverteilung," by Gretory P. Tschebotarioff, *Bautechnik-Archiv*, Heft 8, W. Ernst and Son, Berlin, Germany, 1952.

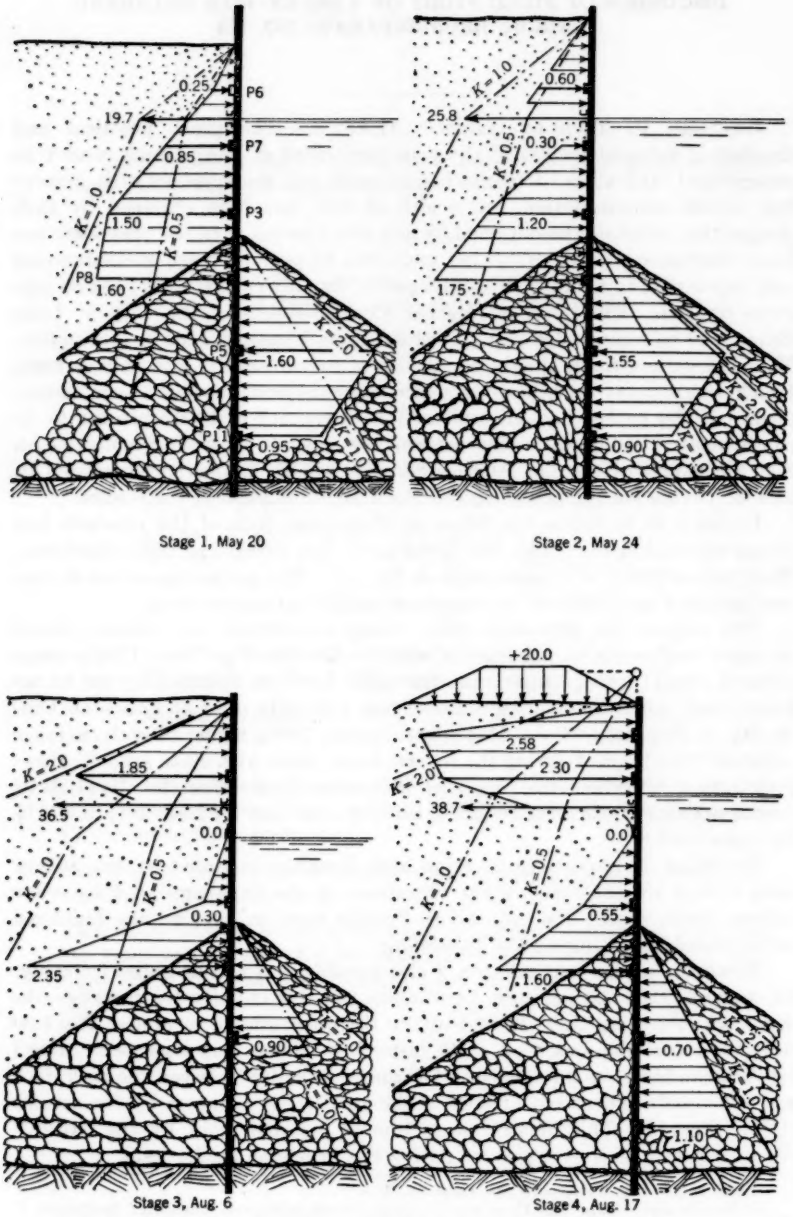

FIG. 18—ANALYSIS OF THE LONG BEACH PIER C BULKHEAD TEST DATA 1949

The term "true arching" means a condition under which the sand grains wedge themselves between unyielding supports in such a manner as to form sand "arches" which can carry part, or all, of their own weight in addition to superimposed loads. Lateral pressures can be increased appreciably thereby.

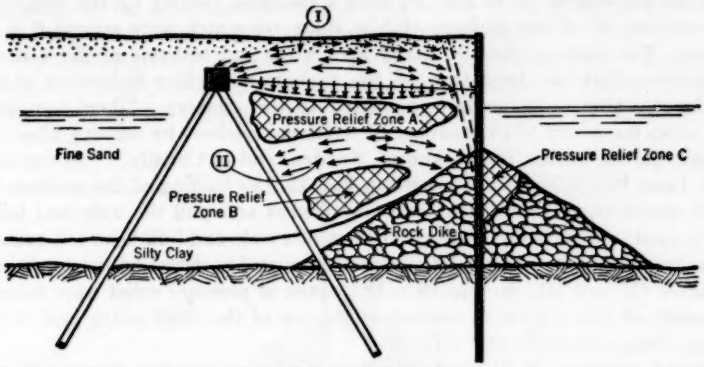

FIG. 19.—ZONES OF "TRUE ARCHING" AND OF PRESSURE RELIEF IN THE SOIL, LONG BEACH PIER C

Their estimation becomes possible from the theory of solid arches.[12,13] "True arching" in sand is facilitated by the yielding of underlying layers. Therefore, the consolidation of the freshly deposited layer of silty clay may have provoked, at first, partial arching of the "true" type above the clay, as shown by the curves (II) in Fig. 19, which arching caused the temporary high pressures recorded by the cell P-4 during stage 1, Fig. 18.

However, it should be noted that "true arching" is a relatively unstable phenomenon in dry, or in submerged, sand. Capillary saturation lends it stability.[12] Therefore, further consolidation of the silty clay layer is likely to have caused the complete redistribution of pressures recorded during the subsequent stages (stage 2, stage 3, and stage 4), shown in Fig. 18.

The shape of lateral earth pressure curves during stages 2 through 4 has only a superficial resemblance to the shape of the theoretical curves proposed by some engineers[14,15] and attributed to the deflection of the sheet piling. There is a fundamental difference. Whereas both the Danish rules[14] and Karl Terzaghi,[15] Hon. M. ASCE, indicate a maximum pressure immediately under the anchor and a minimum at midspan further down, exactly the reverse was observed at Long Beach during stages 2 through 4, Fig. 18. The pressure under the anchor was zero, and increased from there on downward. Thus, this redistribution was produced by causes other than "vertical arching," induced by the deflection of the piling between the level of the anchor and the level of its lower support near the dredge line.

The suggested explanation seems to agree with the views expressed by the author. All hydraulic fills tend to settle somewhat. At Long Beach, the

[13] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, p. 274.

[14] "Normer for Vandbygings-Konstruktioner," *Udgivet af Dansk Ingeniorforening*, Copenhagen, Denmark, 1937.

[15] "Theoretical Soil Mechanics," by Karl Terzaghi, John Wiley and Sons, Inc., New York, N. Y., 1943.

settlement of the upper layers of the fill must have been accentuated by the presence of a layer of silty clay from 5 ft to 6 ft thick, which was deposited ahead of the advancing sand fill proper. This settlement of the fill after placing must have produced some transverse horizontal arching of the type shown by the curves (I) in Fig. 20, with a resulting loading by the weight of the overlying fill of the anchors of 3-in. diameter which were spaced 6 ft on centers. The damage done to many lead wires and to some of the Carlson strainmeters that had been torn off the anchors is another indication of the considerable transverse loads transmitted to the anchors. There are some other cases on record of overloading of bulkhead anchors by settling fills—for instance, in the harbor of Stockholm, Sweden, and in Canada.[16] In the case of the Long Beach Pier C, as shown in Fig. 19, the loading of the anchors by the fill would have made them deflect somewhat and pull the wale and bulkhead inward. Even a slight inward motion of a wale and bulkhead not exceeding an inch should be sufficient to induce horizontal arching of the type shown by curves (I) and (II) in Fig. 19. The zones of pressure relief were formed as a result of this inward movement of the top of the sheet piling and of the arching along curves (I) and (II).

Special attention is drawn to the fact that a progressive decrease of the passive resistance pressures of the outer rock dike against cell P-5 was observed during stages 1 through 4 (Fig. 18), indicating a zone of pressure relief (zone C) which is shown in Fig. 19. This observation is in direct contradiction to any possible assumption of arching in the vertical direction because the outer dike would then have had to act as an outer abutment to any such vertical arches, and to provide part of the necessary increased resistance to their load. However, the observed decrease of passive resistance is entirely consistent with the explanation of horizontal arching above the anchor level which followed the loading and pulling in of the anchor, and through it, of the bulkhead. This movement decreased the passive resistance pressures of the outer dike but increased them on the inner dike. The great increase of the anchor pulls themselves (42% between stages 2 and 3, and 85% between stages 1 and 3) provides further support to the explanation proposed.

Thus, the findings of the Princeton tests concerning the absence of vertical arching behind backfilled bulkheads[11,17] have been confirmed fully by the Long Beach field observations, just as the writer's Princeton tests have been confirmed further by model tests of a somewhat different type in England.[18,19] New important data have been obtained at Pier C, which emphasize the need for the protection of anchors by special measures, such as hollow pipes.[20]

Further field observations are essential. However, such observations alone cannot form the basis for a rapid advance of knowledge concerning the

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[16] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, pp. 511-516.

[17] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951.

[18] "Anchored Sheet-Pile Walls," by Peter W. Rowe, *Proceedings*, Inst. of C. E., Pt. 1, London, England, January, 1952, pp. 27-70.

[19] Discussion by Gregory P. Tschebotarioff of "Anchored Sheet-Pile Walls," by Peter W. Rowe, *ibid.*, September, 1952.

[20] "Soil Mechanics, Foundations and Earth Structures," by Gregory P. Tschebotarioff, McGraw-Hill Book Co., Inc., New York, N. Y., 1951, Figs. 16-29, and 16-30.

actual performance of earth-retaining structures. The number of factors that affect the performance of such structures is so great that, in accordance with the theory of probability, little possibility exists of variation of these factors among observed structures in a manner permitting rapid numerical evaluation of the separate influence of each factor. Model tests, checked by, and correlated to, field studies are essential to that end.[21]

The following further suggestions are made concerning possible future field studies: (*a*) The formation of a silty clay layer should be prevented during future hydraulic backfilling. (*b*) Reliable reference points should be provided

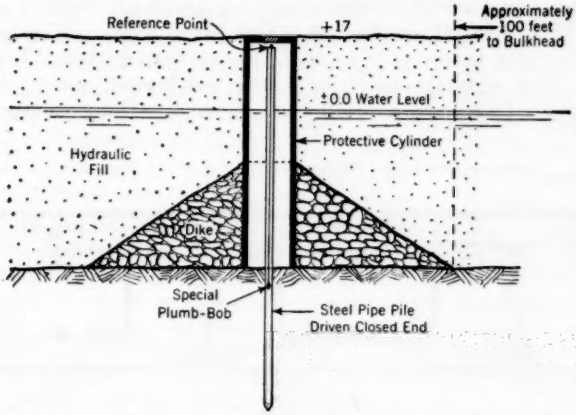

FIG. 20.—POSSIBLE METHOD OF PROVIDING REFERENCE CONTROL POINTS FOR BULKHEAD DISPLACEMENT MEASUREMENTS

for displacement measurements. (*c*) The effects of possible overloading of the anchor piles should be considered. (*d*) Pressure cells might be placed in groups to obtain more accurate measurements.

Prevention of a silty clay layer, mentioned as condition (*a*), may not be easy, but the objective may possibly be attained by providing overflows at regular intervals along the pier to shorten the path of flow of water laden with silt and clay over the pier areas to be backfilled. Relative to suggestion (*b*), the difficulties encountered on this particular job seem to duplicate difficulties encountered elsewhere, when it usually proved impossible to obtain sufficiently accurate displacement measurements by sighting over considerable distances. Therefore, it is suggested that some arrangement, such as that shown by Fig. 20, might be attempted. A steel pipe pile driven closed end into the underlying sand layer would be left unfilled to permit pumping out. An outer protective cylinder—made, for instance, of concrete pipe having a 10-ft diameter—would be placed around it. A conical rock dike could be used to protect the outer cylinder and the soil around the pipe pile from displacements that might be caused later by temporary one-sided loading by the

[21] "Some Unsolved Problems of Importance for the Design of Earth Retaining Structures," by Gregory P. Tschebotarioff, *Bulletin No. 33*, Permanent International Assn. of Navigation Congress, 1950.

hydraulic fill. If any such displacements did occur later, they should involve a tilting of the pipe pile but not a displacement of its lower end. Any such tilting could be measured by a specially designed plumb bob. Two reference points of this type, placed approximately 100 ft from the bulkhead, could be used to check each other and should permit quite accurate displacement measurements of the bulkhead. Regarding deflection measurements themselves, the use of stainless steel strips fixed to the bulkhead sheet piling might serve to decrease some of the corrosion difficulties encountered at Pier C.

Suggestion (*c*) involves possible overloading of the anchor piles. If the anchors were protected by concrete pipes, as the writer has described elsewhere,[20] it is possible that horizontal arching would still develop below them, as shown by curves (II) in Fig. 19. This would overload the outer row of timber piles—the batter piles at Long Beach were spaced 3 ft on centers. Therefore, they certainly would have been overloaded considerably by any such arching. Cases are on record in which something similar happened.

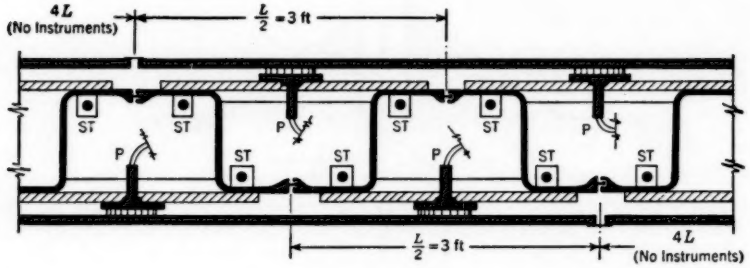

FIG. 21.—SKETCH SHOWING POSSIBLE METHOD OF MOUNTING THE CARLSON STRESSMETERS (P) AND THE CARLSON STRAINMETERS (ST)

Therefore, the question arises as to whether or not it would be advantageous to use a different system of anchorage—such as is illustrated elsewhere by the writer. Precast reinforced concrete anchor blocks might be dropped to the harbor bottom and anchored to the wale by steel anchors. The slope of the inner rock dike might be made to coincide with the slope of the anchors in order to avoid the loading of the latter by the settlement of any underlying fill. The outer rock dike might possibly be omitted entirely under such conditions.

The difficulties encountered during the earth-pressure measurements made by means of pressure cells at Long Beach Pier C seem to indicate that this type of measurement does not have all the advantages sometimes attributed to it. It is not enough just to measure a pressure directly—one has to get an accurate value and that is far from easy. As stated in suggestion (*d*), a possible improvement might consist in using pressure cells in groups, as shown by Fig. 21, so that the cells P would receive much greater total pressures from the individual steel plates that they supported. Similar arrangements have been successfully used in Germany on massive retaining walls. The same general arrangement might be used to make bending strain measurements on the sheet piles by means of Carlson strainmeters, marked ST in Fig. 21. The writer

believes that the strain measurements are more important than the direct pressure measurements because the bending strain readings can be taken all along the depth of the bulkhead down to the original ground level. A bending moment curve thus obtained clearly indicates whether or not there is fixation at the ground level and where the point of contraflexure actually lies. No definite conclusions on this essential point can be made on the strength of the pressure measurements at Pier C described by the author.

All the suggestions in this discussion would require greater sums of money than were available in the case of the author's studies. The writer hopes that this type of work will not only be continued but will be developed.

One reason why research has not yet been developed in civil engineering as much as in some other branches of technology (for instance, in electrical engineering) is the fact that there are no comparably large concerns in the civil engineering industry which could bear the high cost of continued organized research. In Europe, the construction industry contributes to research. The Laboratoires du Bâtiment et des Travaux Publics, Paris, France, perform civil engineering research for the entire construction industry in France, which in return contributes 0.3% of its business turnover. Of course, in Europe the institution of consulting engineers is not so developed as it is in the United States, and most general civil engineering contractors make their own designs, thereby having a greater direct interest in the results of related research than is the case with American contractors. The extent of centralization found in France would not be desirable in a country as large as the United States. Several cooperating, but independent, research centers would produce a valuable spirit of competition. Their location at universities should safeguard their independence from harmful pressures by vested interests of all types.

WALTER C. BOYER,[22] A.M. ASCE.—Those engineers interested in the design of flexible sheet-pile bulkheads should study carefully this paper by Mr. Duke. This report verifies important concepts that have been proposed by other engineers and gives quantitative evidence of factors such as the effect of settlement on tie-rod tension.

It is hoped that this paper will provide the impetus for a series of tests on similar installations that pose a variety of problems. The writer concurs in the statement by Mr. Duke that the problems must be investigated in the field. The bulkhead selected for this test was an excellent choice in one respect and an unfortunate choice in another. The bulkhead provided valuable field evidence of the value of a rock dike in reducing pressures on the wall, but correspondingly reduced the possibility of detecting passive resistance values. This difficulty could have been eliminated, to some degree, by taking strain measurements on the sheet pile. The correlation of pressure readings with strain measurements would, in itself, constitute an interesting study. Although this does not constitute a criticism of the investigation, it adds emphasis to the writer's contention that field investigation is a source of much needed data for the designer of flexible bulkheads retaining cohesive backfill and anchored in cohesive material.

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[22] Asst. Prof., The Johns Hopkins Univ., Baltimore, Md.

The clear description of the instrumentation utilized for this program of study is a welcome contribution—it should serve as a valuable guide to future investigators. There appears to be no unanimity of agreement that the answer to such field investigation lies in electrically controlled equipment. A number of failures, while attempting to utilize such equipment, have been reported. Hence the methods used are worthy of detailed study. However, with the realization that a large financial burden is involved in test programs of this character, duplicate mechanical strain equipment should be utilized wherever possible. Therefore, it would appear that an access well to the tie rod could have been installed and waterproofed, without great expense. This would permit a method of taking periodic tie-rod tension readings for an indefinite length of time.

The development of wall pressures, as presented in Fig. 12, appears to agree with the distribution predicted by the Rankine theory. The deviation from predicted values, after May 24, becomes quite pronounced. The redistribution has been attributed to a settlement of the fill below the tie rods. The development of high tensile stresses, as a result of fill settlement, including actual anchorage failures, have been reported, but no data permitting a quantitative measure have been available. It would be of interest to investigate the loadings which can produce such tension increases.

The steel sheet piles shown in Fig. 6(*e*) are 78 ft long and are anchored by 3-in. tie rods on 6-ft. centers. The tie rods may be taken as 62 ft in length, with a tie-rod support system dividing this span in half. If the rod is considered as a flexible member of little rigidity, the change in length under uniform loading may be determined from the expression:

$$\Delta l = \tfrac{1}{2} \int_0^l \left(\frac{dy}{dx} \right)^2 dx \qquad (1)$$

in which l is half of the tie-rod length, y is the deflection, and x is the distance from the center support—the coordinate system origin assumed to be at the center support. If the tie rod is taken as a beam, with hinged ends, and supported at the center, the deflection is

$$y = \left(\frac{5}{48} w\, l\, x^3 - \frac{w\, l^2\, x^2}{16} - \frac{w\, x^4}{24} \right) \frac{1}{E\, I} \qquad (2)$$

in which w is the uniform load, E is the modulus of elasticity, and I is the moment of inertia of the bulkhead section.

From Eqs. 1 and 2,

$$\Delta l = \frac{1}{2} \int_0^l \left[\left(\frac{5}{16} w\, l\, x^2 - \frac{w\, l^2\, x}{8} - \frac{w\, x^3}{6} \right) \frac{1}{E\, I} \right]^2 dx \qquad (3)$$

The tie-rod tension has been determined by Joel I. Abrams, J. M. ASCE, by equating Δl to the change in length caused by P, the change in axial force.

This is expressed as

$$\frac{P\,l}{A\,E} = \frac{1}{2}\int_0^l \left[\left(\frac{5}{16}\,w\,l\,x^2 - \frac{w\,l^2\,x}{8} - \frac{w\,x^3}{6}\right)\frac{1}{I\,E}\right]^2 dx \quad \ldots\ldots (4)$$

in which A is the area of the section.

For the problem under consideration, Eq. 4 reduces to

$$w = 0.22\sqrt{P} \quad \ldots\ldots (5)$$

and if some yielding of the anchorage or wall is to be considered, Eq. 5 becomes

$$w = 0.22\sqrt{\frac{100\,P}{Y}} \quad \ldots\ldots (6)$$

in which Y is the yield defined as a percentage of Δl.

These relationships neglect the moment caused by original tie-rod tension and the moment developed by an increase in tie-rod tension. However, for a member as flexible as a tie rod, such an assumption should not influence the results greatly. The resulting load on the tie rod, due to settlement, is not considered to be uniform, but this assumption leads to an indication of the loading requirements to produce increases of tie-rod tension as reported in the paper.

By use of Eq. 6, the values for uniform loading to produce changes in tie-rod tension as given in Table 6 may be computed. It is noted that a yield of 50% does not alter the values greatly.

TABLE 6.—Tie-Rod Tension Developed Under Uniform Loading, Tie Rod Acting as a Beam

Date	Tie-rod tension T, in kips	Change in axial force P, from May 24, in kips	Uniform load w, in lb per ft, Eq. 5	Uniform load w, in lb per ft, for Y = 50%, Eq. 6
May 24	155	...	0	0
June 3	182	37	36	51
June 11	205	50	49	69
July 12	209	54	51	73
August 5	214	59	54	76
August 6	219	64	56	79
August 17	229	74	60	85

For a tie-rod spacing of 6 ft, the tributary overburden is approximately 8,800 lb per ft of tie-rod length. It is not implied that a substantial percentage of this loading could act on the cable under settlement conditions, but the values as given in Table 6 are feasible.

If the tie rods are considered to act as flexible cables under an initial tension T_0, the change in length under uniform loading may be determined by the expression[23]:

$$\Delta l = \frac{1}{2}\int_0^l \frac{w^2}{T^2}\left(x - \frac{l}{2}\right)^2 dx \quad \ldots\ldots (7)$$

[23] "Statistically Indeterminate Structures," by L. C. Maugh, John Wiley & Sons, Inc., New York, N. Y., 1946, p. 261.

The final tie-rod tension T can be determined by equating Δl to the change in length due to axial force. This is expressed as

$$\frac{(T - T_0)\, l}{A\, E} = \frac{1}{2} \int_0^l \frac{w^2}{T^2} \left(x - \frac{l}{2} \right)^2 dx \quad \ldots\ldots (8)$$

For the problem under consideration Eq. 8 becomes

$$w = \frac{T}{92{,}500} \sqrt{T - T_0} \quad \ldots\ldots (9)$$

By use of Eq. 9 the values for uniform loading to produce tie-rod tensions as given in Table 7 may be computed. It is noted that the uniform loadings required are considerably greater than those for the condition of the tie rod acting as a flexible beam (Table 6).

TABLE 7.—TIE-ROD TENSION DEVELOPED UNDER UNIFORM LOADING, TIE ROD ACTING AS A CABLE

Date	Tie-rod tension T, in kips	Change in tie-rod tension $T - T_0$ from May 24, in kips	Uniform load w, in lb per ft	Tie-rod tension T', in kips[a]
May 24	155[b]	...	0	155
June 3	182	27	326	163
June 11	205	50	497	172
July 12	209	54	525	174
August 5	214	59	562	176
August 6	219	64	600	179
August 17	229	74	675	183

[a] T' is the tie-rod tension for the tie-rod supports placed at 15 ft 6 in. on centers. [b] This value is the initial tie-rod tension, that is, $T_0 = 155$ kips.

The values presented in Tables 6 and 7 probably bracket the actual loading that was developed because of the settlement of the bulkhead under consideration. The placement of turnbuckles between the supporting piles probably reduces the action of the tie rod to a case intermediate between the case of a flexible beam and a cable.

Although the preceding analyses are admittedly rational, they lead to interesting conclusions relative to the spacing of tie-rod support systems. In the bulkhead design under consideration, the tie rod had an unsupported length of 31 ft (not considering earth support). If this spacing is reduced to 15 ft 6 in., the increase in tie-rod tension is markedly reduced.

For the case of the tie rod acting as a flexible beam,

$$w^2 \propto \frac{P}{l^6} \quad \ldots\ldots (10)$$

Hence, if the spacing of tie-rod supports is reduced to $l/2$,

$$w^2 \propto \frac{64\, P}{l^6} \quad \ldots\ldots (11)$$

Thus, for the existence of uniform loads as previously computed, P would develop values of less than 2% of those measured in the study.

For the case of the tie rod acting as a cable,

$$w^2 \propto \frac{T^2 (T - T_0)}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Hence, if the spacing of tie-rod supports is reduced to $l/2$,

$$w^2 \propto \frac{4 (T')^2 (T' - T_0)}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Thus, for the existence of uniform loads as previously computed, the value of T' computed in accordance with Eq. 13 is given in Table 7. The values of T' are less than 38% of those measured in the study.

This indicates the importance of the spacing of tie-rod supports to insure tension values consistent with the design considerations. Unfortunately, these calculations are possible only as a result of the values developed in the study. It is not possible to estimate, during the design stage, even by rational methods, the uniform loads on the tie rods which may exist under conditions of fill settlement. This problem is worthy of continued study and research. It is important in a number of design problems other than the one under consideration.

In view of the uncertainty of such estimates, it would appear that the tie rods should be placed in conduits which would prevent development of uniform loadings due to the soil overburden. This would be necessary for conditions where relatively large settlements are likely to occur. However, this solves the problem by avoiding it, and it certainly is not the most economically advantageous method. It would appear from the results of the study that the spacing of tie-rod supports at 31 ft, for a fill where settlement is likely to occur, is excessive, and therefore should be reduced to 15 ft.

PAUL BAUMANN,[24] M. ASCE.—Valuable information concerning active and passive earth pressures on a steel sheet-pile bulkhead of unusual height is presented in this paper. Great care and attention to details were given to the planning and installation of the measuring instruments and to the arrangement of the facilities for measuring tie-rod tension and wall deflections. However, a number of features concerning the active and passive pressures remained vague and elusive.

A test on a steel sheet-pile bulkhead was conducted by the writer in the area described by Mr. Duke.[25] Although the two tests have much in common, the principal purpose is different. The writer's test was conducted to ascertain passive pressure or resistance in relation to safe anchorage, whereas the determination of active and passive pressures during, and subsequent to, the fill operation was the principal objective of the test described by Mr. Duke.

In the writer's test the active pressure was created by use of sea water. The active pressure was known above the ground surface and was known approximately below the surface, whereas the magnitude and the distribution

[24] Asst. Chf. Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

[25] "Analysis of Sheet-Pile Bulkheads," by Paul Baumann, *Transactions*, ASCE, Vol. 100, 1935, p. 707.

of the passive resistance were determined from observed deflections and equilibrium conditions. The free height of the bulkhead, over which the active pressure was exactly known and the deflections were precisely measured, was 15 ft and the penetration was 12 ft. Hence, deformations for approximately 56% of the sheet-pile wall could be observed. In Mr. Duke's test the free height of the bulkhead above the top of the rock-fill dike was 26.5 ft, the penetration into the dike was 28.5 ft, and the penetration below the floor of the harbor was a minimum of 22.0 ft. Hence, the exposed length of sheet-pile wall which lent itself to direct observation was approximately one third of the total length.

It is regrettable that pressure cells on the active-pressure side were not extended so far down as on the passive-pressure side prior to the placing of the dike and that the number of cells on the passive-pressure side had to be limited to two, 10 ft apart vertically. This latter limitation caused straight lines to be drawn between the measuring points and the top of the dike as shown in Fig. 12 which, therefore, does not represent the true distribution of passive pressure. In all probability, a curve drawn through the respective three points would have been a better approximation of the pressure distribution. The writer showed [26] the actual distribution of passive pressure for homogeneous, granular materials, based on large-scale tests.

A number of the features shown in Fig. 12 are rather difficult to understand. Among them is the acute increase in active pressure, starting on June 11 below an approximate elevation of -8, accompanied by a decrease in the passive pressure in the four subsequent drawings. On July 12 and August 6 the lowest passive pressure cell either was not read or perhaps it showed erratic readings which had to be ignored. Such mishaps seem to be the rule rather than the exception where earth pressure cells are involved. However, the response of the pressure cells to tidal changes—that is, changes in external water pressure—is remarkable. A rise in tide generally caused an increase in active pressure associated, paradoxically, with a decrease in tie-rod tension and passive resistance. The active pressure, therefore, was actually in part passive pressure.

The writer's statement[27] regarding the impossibility of securing an exact, analytical solution so long as the active loading could not be expressed as a steady function of the height appears to have been confirmed by Mr. Duke. The close spacing of the tie rods, for instance, gave rise to a vertical arching and thereby an approach to the effect of a relieving platform.

In the study of the deflections of the sheet-pile wall, Mr. Duke assumed no deflection at El. -38, the surface of the soil in situ. No support is offered for this assumption and, on the basis of Fig. 12, its validity must be doubted. Indication of the author's own doubt appears evident in item 9 (under the heading, "Suggestions for Future Bulkhead Field Studies")—

> "Creative thought is needed on the problem of the difficult and probably expensive quantitative observation of passive resistance phenomena at the base of a bulkhead."

This is a sound suggestion.

[26] "Analysis of Sheet-Pile Bulkheads," by Paul Baumann, *Transactions*, ASCE, Vol. 100, 1935, p. 728.
[27] *Ibid*, p. 738.

It would be interesting if tests similar to those conducted by Mr. Duke and the writer could be broadened so as to include sheet-pile bulkheads of at least two greatly differing rigidities. On the basis of the statically indeterminate solution applied to two such bulkheads—a steel sheet-pile wall and a reinforced concrete sheet-pile wall—the writer showed[28, 29] that the latter, because of its greater rigidity, may require a penetration approximately 20% greater than that of the former. It is well to recall that bulkhead failures have, almost without exception, been associated with faulty anchorage rather than deficient structural qualities.

S. PACKSHAW,[30] M. ASCE.—Opportunities for studying the behavior of retaining walls constructed of sheet-steel piling are rather rare. The new wall is frequently driven in front of an existing structure, which has an unknown effect on the earth pressures and the stresses which the piling has to withstand. It may also be difficult to determine the properties of the soil and of the backfilling with enough accuracy to assess their precise active and passive pressures. The conditions at the Long Beach Harbor bulkhead were ideal for a field study such as that undertaken by Mr. Duke, and some valuable deductions can certainly be made. A study of the results indicates, however, that the information yielded by the tests is applicable primarily to bulkheads tied to anchorages on closely spaced raking piles; it would be misleading to apply the author's conclusions to sheet-pile retaining walls in general.

It is assumed that the soil properties given in Table 1 were determined by the usual soil mechanics procedures. The properties are fairly typical of materials used for backfilling a wall and of fine sand into which piling is driven. The corresponding Rankine or Coulomb active pressure coefficients for the sand strata (neglecting friction between the soil and the wall) vary from approximately 0.24 to approximately 0.28, as compared to the lateral pressure coefficient of 0.7 suggested by Mr. Duke. Many designers would use the Rankine coefficients, or values close to them, as a basis for the computation of a retaining wall for similar conditions. The corresponding pressure diagram, which incorporates some minor approximations and assumes that an angle of wall friction of 20° is developed on the passive side, is shown in Fig. 22(*a*). The dotted lines superimposed on the conventional pressure diagram serve as a comparison to the author's measurements shown in Fig. 12. The difference between the lines is too great to be accounted for merely by errors in ascertaining the soil properties and earth pressure coefficients. The explanation must lie elsewhere.

Fig. 12 shows a gradual, although not very regular, increase in active pressure until May 20, when backfilling was nearly complete. After that date, the pressure above the tie rods continued to increase throughout the period of observation, but below the tie rods the pressures fell off sharply. It is probable that the increase in pressure above the tie rods and the reduction in pressure below the tie rods were caused primarily by horizontal arching within the soil between the bulkhead and the anchor piles. Measurement of the tie-rod loads

[28] "Analysis of Sheet-Pile Bulkheads," by Paul Baumann, *Transactions*, ASCE, Vol. 100, 1935, p. 794.
[29] *Ibid.*, p. 795.
[30] Director, British Steel Piling Co., Ltd., London, England.

shows them to be about three times as great as might be expected from the conventional "fixed-earth support" computation of Fig. 22(*a*). Abnormal stresses were developed when the fill under the tie rods consolidated, thus leaving the tie rods with little or no support for carrying the weight of the fill above them. The tie rods should have been laid along the inverts of concrete pipes, the diameter of the pipes being large enough to take up any settlement of the fill, thus protecting the rods from vertical loads for which they were not

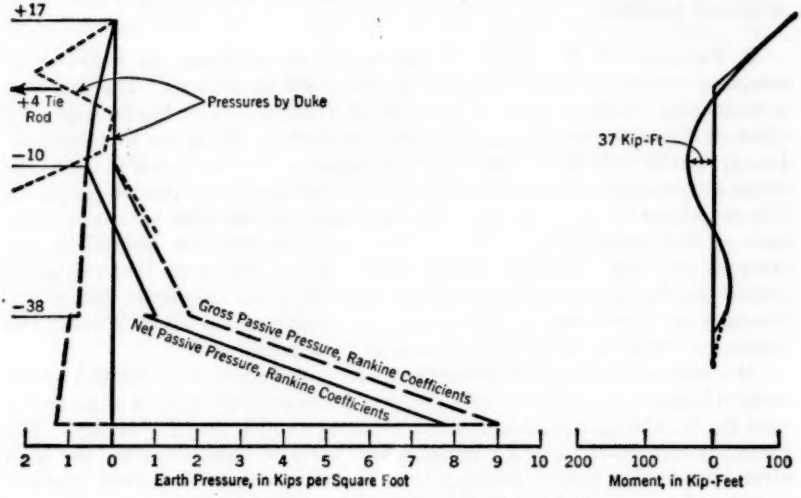
+17
+4 Tie Rod
Pressures by Duke
−10
−38
Gross Passive Pressure, Rankine Coefficients
Net Passive Pressure, Rankine Coefficients
37 Kip-Ft
2 1 0 1 2 3 4 5 6 7 8 9 10
Earth Pressure, in Kips per Square Foot
200 100 0 100
Moment, in Kip-Feet

FIG. 22.—PRESSURES AND MOMENTS, STATION 27+30

designed. This simple method has been adopted elsewhere and is quite effective. The tie rods, however, are too small and too far apart to act as a relieving platform, even though the observations on and after July 12 showed that the pressure immediately below the ties had been reduced to zero—a condition typical of pressure diagrams for walls with relieving platforms.

It seems more likely that this reduction to zero pressure was caused by arching within the wedge-shaped soil mass contained between the bulkhead and the forward raking piles, because these piles are so close together that they can be regarded as a continuous wall. European practice rarely utilizes this type of construction, but when it does, the piles are more heavily loaded and consequently spaced wider apart. If the backfill does not permit the use of a normal type of anchorage, such as concrete blocks, groups of short vertical sheet piles, or an anchor wall, it is usual to secure the wall to a relieving platform on raking piles.

It is regrettable that the mechanical strain gages for measuring the bulkhead bending moment proved to be unreliable and that no observations of the stress in the sheet piles were obtained. The conventional method of analysis, shown in Fig. 22(*b*), is to assume that the waling load is approximately 9,400 lb per ft of wall and that the bending moment is approximately 37 kip-ft when

the backfill is level with the top of the wall. In designing the wall, allowance must be made for the surcharge caused by filling to El.+20 and for the groundwater level remaining 2 ft or 3 ft higher than the harbor water level at low tide. When all this is taken into consideration the bending moment and waling load make necessary the choice of a section of steel piling and size of tie rods in accordance with those selected by the designer of the bulkhead. Innumerable retaining walls have been designed on the same basis, which, for all its shortcomings, has been proved in practice to be satisfactory for normal conditions. Mr. Duke's observations do not appear conclusive enough to warrant the general use of the lateral pressure coefficient of 0.7 instead of the Rankine coefficients corresponding to the soil properties, or of the two rules which are listed under the heading, "Bulkhead Performance During Filling: Lateral Pressures in Dike."

The large deflections which were observed are approximately three times greater than would be expected, but they may have been influenced by the elastic extension of the tie rods (about $\frac{3}{4}$ in. under maximum load) and by the probably even greater movement caused by the consolidation of the ground at the anchorage. However, the wedging and arching of the soil mass between the wall and the front raker piles, and the consequent concentrations of pressure, are likely to be the main reason for the large deflections.

Researches on model flexible walls by Mr. Tschebotarioff,[31] and P. W. Rowe[18] have done much to advance the knowledge of the behavior of these walls. Mr. Rowe's research led to the result that fixity can be expected to develop when the ground below the dredging level is composed of dense sand or similar material, with an angle of internal friction approximately equal to 40°. This appears to justify the conventional method of Fig. 22(*b*). Mr. Rowe also showed that, with all but the more rigid sections of sheet piling, the actual bending moment is less than that computed directly from the earth pressures on the wall. He also obtained the quantitative relation between the bending moment, the flexural properties of the material, and the section of the sheet piling.

If, for example, the calculation for the Long Beach Harbor bulkhead showed that the stress in the MZ-38 section, which has a section modulus of 46.8 in. cubed per ft of wall, is 16,000 lb per sq in., then the stress in the more flexible MZ-32 section will not be 19,500 lb per sq in., because of its section modulus of 38.3 in. cubed per ft of wall, but some substantially lower figure. If Mr. Rowe's method is compared with the conventional fixed earth-support procedure, it is found that the maximum bending moment may be greater or smaller, depending on the ground conditions and the type of piling. The difference between methods is frequently small, but one of the advantages of Mr. Rowe's method is that it clearly demonstrates the effect of varying the section of piling and using steels with different permissible stresses.

The results of Mr. Duke's research do not lend themselves to universal use in the design of sheet-pile retaining walls. However, two vaulable deductions can be made: First, the tie rods are subjected to large stresses if they

[31] "Large-Scale Model Earth Pressure Tests on Flexible Bulkheads," by Gregory P. Tschebotarioff, *Transactions*, ASCE, Vol. 114, 1949, p. 415.

have to carry the weight of the superimposed fill caused by the settlement of the ground below them; and second, the unpredictable concentrations of pressure may result from the effect of wedging and arching of the soil between the wall and the closely-spaced raking anchor piles.

W. F. WAY,[32] M. ASCE.—Worthwhile research can be successfully performed in the field, as is shown by the useful results of an interesting test by Mr. Duke.

The specific tests were well planned, executed, and presented. All that was planned did not work out as was desired and this is to be expected. In the test, the planners were pioneering in the sense that they did not have the benefit of the results of similar tests which might highlight the important features to be considered. Engineers interested in and conversant with problems peculiar to sheet-pile bulkhead design should find much of interest in this paper. Engineers who are to design a sheet-pile bulkhead will find much help, even though the design in question might be substantially different from that of the test. A study of the tests and their results will assist the designer to "feel" his way to a better design.

Mr. Duke's "Suggestions for Future Bulkhead Field Studies" is a well-considered listing of important items. Item 8 is very important and especially true because it includes tests which cannot be made in the laboratory. Since the function of a bulkhead is to retain a fill on which a live load can be carried, it is impossible to overemphasize the need for study of the effect of surcharge. The pressures and stresses in the structure caused by a surcharge load will vary in accordance with the lateral pressure coefficient of the fill behind the bulkhead. The first application of a surcharge load will increase the tie-rod stresses; after removal of the surcharge, there will be a removal of most of the stress, but a residual stress will remain in the tie rods. Subsequent loadings and removals will follow the same cycle. Studies to determine the increased stresses, rebounds, and residual stresses caused by these successive loadings would be valuable.

Field tests to check design assumptions have not been extensively included in engineering research. There seems to be hesitancy among designers to promote field tests such as those described by Mr. Duke. The reasons for this attitude are:

1. The designer is not equipped for testing, nor, perhaps, has he had the experience to enable him to undertake such tests.
2. Almost any field test would interfere with the progress in construction, which would adversely influence construction personnel.
3. Tests require funds, and finding someone to furnish these funds requires both time and patience.
4. There is a feeling among many designers that the tests apply only to a few particular sections, and the results obtained are not representative, and therefore not worthwhile.

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[32] Vice-Pres., Frederic R. Harris, Inc., New York, N. Y.

With the knowledge of these objections to field tests, the writer feels that there are instances when the reasons for making the tests outweigh the objections. Advantage should be taken of these favorable instances as they occur, and Mr. Duke is to be commended for having the interest and energy necessary to see this interesting field test made.

J. Owen Lake,[33] A. M. ASCE.—The opportunity for scientific study of the structural performance of full-size anchored sheet-pile bulkheads is rare. It is therefore regrettable that Mr. Duke was prevented from obtaining unqualified data by the doubtful validity of the readings obtained from the stressmeters located in the dike and by the progressive failures in the lead wires to the strainmeters. The author stated, under the heading, "Instrumentation: Soil Pressure," that "The probable validity of measurements in the dike was questioned from the first because of the large size of dike fragments." Therefore, cannot the authenticity of the free body diagrams in Fig. 12 be doubtful below El. + 0.7? The stressmeters were installed only at Station 27 + 30 and were vertically disposed in a single line, and from these few measurements the distribution and intensity of the lateral earth pressure was deduced. However, reference to the tie-rod tensions listed in Table 2 indicates a wide scattering of values which could mean a variation of pressure in a horizontal direction at the face of the bulkhead, although a large part of this variation in the tie-rod loading is probably due to the differential yield of the closely spaced ties combined with the continuity of the wale. Nevertheless, considerable local variation of pressure is expected behind bulkheads and this necessitates the averaging of readings from a substantial number of stressmeters if a representative diagram of pressure distribution is to be obtained.

In Fig. 5 it is shown that silty clay existed below El.—35 in borings A-1 and A-2, which were sunk after the filling operation was complete. These borings were adjacent to the test stations, and this silty clay stratum may not have been entirely displaced by the deposition of the rock dike. Because of the low cohesion of silty clay in the initial stages of filling, there would be a horizontal plane of low shear resistance at El.—35 which would allow appreciable forward deflection of the bulkhead during the early stages of filling. This is especially true since there is a large tidal lag. For example, on April 23 at Station 30 + 30, with a fill increase of only 2 ft, the bulkhead deflection increased from 0.16 in. to 0.86 in., an unexplained increase unless the significance of the tide at El. 5.1 on the inside and El. 1.9 on the outside is acknowledged. The measured deflection profiles were obtained only to El.—9.5, and it is noted that they could easily approximate computed deflection curves if the bulkhead had been displaced forward at El.—38 during the earlier stages of filling. If such movement had taken place, the assumption that the bulkhead is fixed against rotation at El.-38 would be incorrect. Some confirmation of a lower point of "fixity" is possibly indicated since the predictions of the deflections based on the soil pressure measurements were 25% too low. The increase of pressure above the wale and the decrease of pressure below the wale have been attributed mainly to the settlement of the fill, but it is suggested that if forward

[33] Cons. Engr., Toronto, Ont., Canada.

movement at El.−38 occurred with time, caused by soil creep on the passive side, the bulkhead would tend to rotate about the tie-rod level and would result in the variation of pressure that was observed at Station 27 + 30 without any appreciable alteration to the deflection curve above the reference level of El.−9.5. Variability of strata below the harbor bed level of El.−38 could then account for the absence of pressure variation at Station 30 + 30.

The phenomena of decreased lateral pressures on the bulkhead below the tie rods could also be associated with the reduction of excess pore-water pressures as consolidation proceeded. Such excess hydrostatic pressures would be unlikely to occur appreciably above the tie rods where the sand is of medium particle size. However, below the ties the sand is fine and contains a proportion of silt, and such a soil would undoubtedly be in a loose state just after deposition. During deposition, by hydraulic fill, the soil mass acts as a viscous fluid and therefore exerts relatively high lateral pressures that approach a lateral pressure coefficient of unity. Thereafter the effective stress develops as the pore-water pressure falls and the lateral earth pressure coefficient also is reduced. Thus, as further hydraulic fill is deposited on the soil already in position, the bulkhead tends to deflect and inevitably brings into effect the reduced lateral earth pressure coefficient which is operative in the lower regions of the fill where the "effective" stress has developed appreciably. In Table 3 it can be seen that the stressmeters at El.−7.3 and El.−14.9 did not indicate a linear increase of pressure as the fill rose from El.−7.0 to El.+5.8.

A further factor which could undoubtedly contribute to the relatively high lateral pressure coefficient of 0.7, deduced from the observations, is the tidal lag which occurred frequently during the filling operations. During periods of tidal lag the deflection of the bulkhead would be increased and the sand fill would move forward correspondingly. As the tidal lag reduced, the bulkhead would be left in a prestressed condition, resulting in a tendency toward the development of passive resistance in the fill immediately behind the bulkhead. If the tidal lag had been large enough, actual passive pressure would have been obtained after the tidal lag had been dissipated. Therefore, if after the fill was complete, the nuts of the tie rods were slackened to allow the bulkhead to move forward slightly at the waling level, the relatively high lateral earth pressure would be reduced. This expedient is worthwhile, as the load of 200 kips in some of the ties appears to give a factor of safety only slightly greater than unity when referred to the yield stress.

Under the heading, "Summary of Test Results," Mr. Duke states that

> "The surcharge piled close to the bulkhead after completion of filling was carried principally by the tieback system, pressures below the wale being not much affected."

It is suggested that precisely this appearance of no increase in lateral pressure below the wale could occur because of a prestressed condition of the bulkhead caused by tidal lag during filling operations and resulting in a lateral pressure coefficient considerably above the normal active value for sand. Such a condition is then obtained whereby a certain degree of surcharge can be added without a noticeable change in the lateral pressure below the wale until the

intensity of the surcharge results in lateral pressures greater than the pressures arising from the restitutional force of the prestressed condition of the bulkhead.

It is agreed that settlement of the fill and the partial support of the upper part of the fill on the tie rods is always an important consideration. It may be that this condition had an appreciable influence on the field study under discussion. However, the complexity of the subject is perhaps further exemplified by the existence of the factors mentioned by the writer.

K. TERZAGHI,[34] HON. M. ASCE.—Observations such as those described in the paper are urgently needed and their merits are too obvious to require special emphasis. Therefore, this discussion deals exclusively with the controversial aspects of some of the findings.

Coefficient of Lateral Earth Pressure.—Of the author's conclusions, the most startling is contained in a statement under the heading, "Summary of Test Results," that: "The proportionality constant, or coefficient of lateral pressure, was found to be about 0.7 ***." Previously, it has been assumed that the coefficient of lateral sand pressure K on bulkheads, like that of the sand pressure on retaining walls, is slightly greater than that of the coefficient K_A of active earth pressure; and all the experimental investigations made prior to publication of this paper are in accordance with this assumption. The value of K_A is less than 0.3. If a bulkhead were exceptionally rigid and the anchorage did not yield at all, the value of K could conceivably increase to 0.4 or 0.45. The author's statement that K was equal to 0.7 would imply that the value K of the coefficient of lateral earth pressure of sand may range between the wide limits of less than 0.3 (coefficient of active earth pressure) and as much as 0.7. If K could, in fact, assume any value within this range, the attempts of engineers to establish rational methods for bulkhead design might as well be abandoned as hopeless. Considering this serious implication of the author's findings concerning the value of K, it is of utmost importance to ascertain to what extent the value 0.7 can be relied upon.

The author's evaluation of K is based on two independent sets of data, the results of pressure cell readings and those of the measurement of the sheet-pile deflections. He states that both sets of data lead to the same conclusion, that K equals 0.7.

The reliability of the pressure cell data depends on the accuracy of the calibration curve. A description of the procedure for calibrating the cells can be found under the heading, "Instrumentation." The results of the calibration are indicated in Fig. 9. In Fig. 23(*a*), which is a slightly modified replica of Fig. 9, the water pressure curve is labeled W and the soil pressure curve, S. The abscissas represent the unit pressure on the contact face of the cell, and the ordinates, the change in the resistance ratio.

Curve W shown in Fig. 23(*a*) represents the relation between the change in the resistance ratio and the pressure exerted by water on the contact face, whereas curve S represents the relation between change in the resistance ratio and the soil pressure. The position of curve S with reference to curve W depends on the calibration procedure. Since it cannot be taken for granted that

[34] Prof., Civ. Eng., Div. of Eng. Sciences, Harvard Univ., Cambridge, Mass.

the author's test arrangement represents a satisfactory equivalent for the field conditions, it is necessary to inquire whether or not his calibration curve S is compatible with the operation of the cells attached to the bulkhead.

The position of the soil pressure curve, S, in Fig. 23(*a*), with reference to the water pressure curve W has the following physical meaning. If the contact face of the cell were acted upon, for instance, by a water pressure of 0.8 kips per

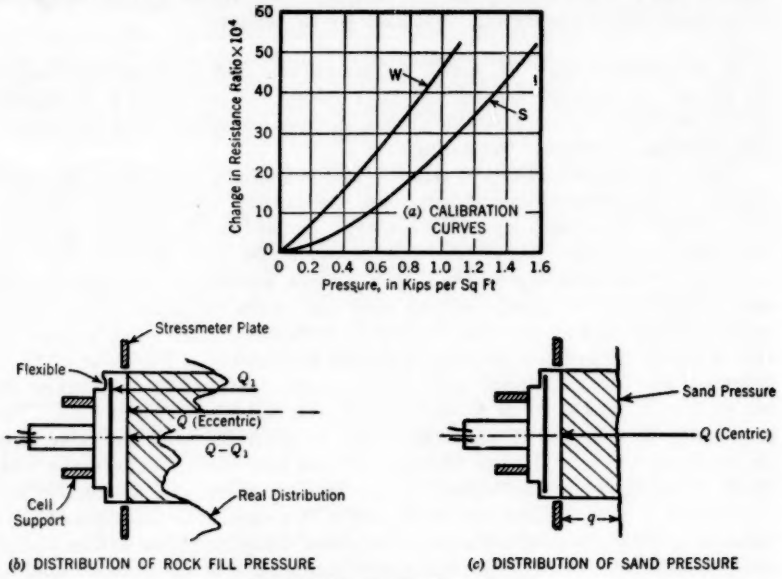

FIG. 23.—STRESSMETER CALIBRATION

sq ft, the strainmeter would indicate, according to Fig. 23(*a*), a change in the resistance ratio of 36×10^{-4}. Since the radius r of the contact face is approximately 0.30 ft, the total pressure on the contact face would be

$$Q = 0.8 \times r^2 \pi = 0.23 \text{ kips} \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

However, when the cell was acted upon by the same load, Q, in the testing machine, during the test that was supposed to simulate the loading conditions in the field, the change in resistance ratio was only 18×10^{-4}, corresponding to a water pressure of only 0.46 kips per sq ft and to a value of $Q' = 0.46 \times r^2 \pi = 0.13$ kips.

The resistance ratio increases approximately in direct proportion to the deflection of the membrane shown in Fig. 8, which in turn increases in direct proportion to the pressure q_m in the mercury film contained in the cell. If the contact face of the cell is acted upon by water under a pressure of 0.8 kips per sq ft, the pressure in the mercury film is approximately equal to the water pressure,

$$q_m \approx 0.8 \text{ kips per sq ft} = \frac{Q}{r^2 \pi} = \frac{0.23}{r^2 \pi} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

yet the same load Q, applied to the cell by the testing machine, produced (in the mercury film) a pressure of not more than

$$q'_m = 0.46 \text{ kips per sq ft} = \frac{Q'}{r^2\pi} = \frac{0.13}{r^2\pi} \quad \ldots\ldots\ldots\ldots (16)$$

This pressure is equal to the water pressure corresponding to a change in resistance ratio of 18×10^{-4}.

The importance of the difference between the values q_m (Eq. 15) and q_m (Eq. 16) indicates that the pressure cells had some defects. The disks on both sides of the mercury film were not rigid enough to prevent disturbing deformations, and part of the space between the disks was probably filled with gas—not with liquid mercury. Because of these defects of the cells the method used for calibrating the cells was not appropriate, since the readings on the cells depend largely on the type and magnitude of the cell deformations produced by the application of the test load. If the thickness of the rubber pad had been increased, or if the pad had been made of a softer or stiffer rubber, contrastingly different calibration curves would have been obtained.

The conditions under which the cells operate in the field are illustrated in Figs. 23(*b*) and 23(*c*). Fig. 23(*b*) shows a cell in contact with the rock fill. The rock fill contains 10% of stones larger than 3 in. and the diameter of the cells is approximately 7 in. The distribution of the pressure of a rock fill over an area with a diameter of 7 in. is inevitably nonuniform and involves eccentric loading of the contact face. Because of this loading condition, the inner face of the outer disk of the cell bears at some point against the outer edge of the inner disk. Hence, one part Q_1 of the total load Q acting on the contact face will be transmitted directly onto the inner disk which rests on the stiffeners, and the pressure in the mercury film will be (Eq. 15) $q'_m = \frac{Q - Q_1}{r^2\pi} < \frac{Q}{r^2\pi}$. The value Q_1 depends on the eccentricity of the load, which is different for every cell. This fact excludes the possibility of determining the lateral pressure of rock fill by means of small pressure cells such as those used on Pier C.

Fig. 23(*b*) shows a cell in contact with fine to medium sand deposited by sluicing. The distribution of pressure exerted by such a sand can be assumed almost perfectly uniform over the contact face. The increase with depth of the lateral pressure of the sand is much smaller than the increase of the pressure in the mercury film because of the weight of the mercury. In the writer's opinion there seems to be no reason why the relation between the same pressure of a given intensity and the mercury pressure should be different from the relation between water pressure of the same intensity and the mercury pressure. Hence, the calibration curve for the sand pressure should be practically identical with the calibration curve for water pressure, W in Fig. 23(*a*). If the author had evaluated the strainmeter readings on the basis of curve W in Fig. 23(*a*), he would have obtained a K-value of approximately 0.4. Since he used curves S, he obtained $K = 0.7$.

Whichever calibration curve is used, the interpretation of the strainmeter reading is based on the assumption that the unit pressure on the contact face of the cell is equal to the average unit pressure on the bulkhead at

the elevation of the cell. If the plane of the contact face would yield under the influence of the lateral sand pressure without warping, this assumption would be justified. However, while the sand fill is being placed, the plane recedes slightly between the flanges of the sheet piles and it also yields on both sides of the rigid bridges that support the cells. The consequences of this condition are illustrated in Fig. 24.

Fig. 24(*a*) is a vertical section through the center line of a cell. Since the cell is mounted on a relatively rigid bridge, the stressmeter plate yields above and below the cell under the influence of the increasing lateral sand pressure, with reference to the cell. Fig. 24(*b*) represents the plane B-B of Fig. 24(*c*), in

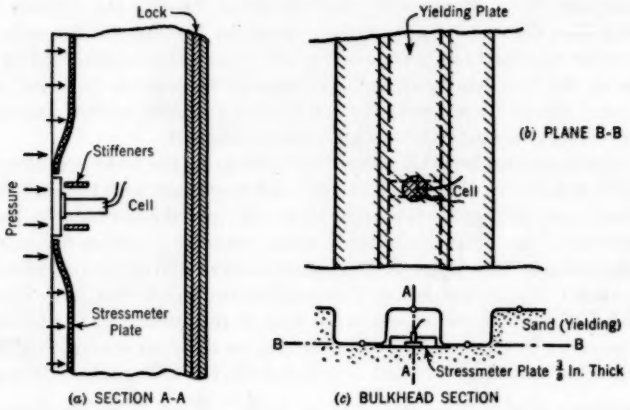

FIG. 24.—WARPING OF THE PLANE CONTAINING THE SOIL CONTACT FACE OF THE PRESSURE CELLS (BLANK AREAS YIELD WITH REFERENCE TO SHADED AREAS)

which the stressmeter plates lie. The shaded areas indicate the relatively unyielding areas of this plane. The yield of the lateral support between the shaded areas involves an increase of the unit pressure on the shaded areas and a decrease of the pressure on the blank areas, with reference to the average unit pressure q on the bulkhead, at the elevation of the cell. The cell is located in a shaded area. Therefore, the unit pressure on the cell will exceed q. The evalualtion of the strainmeter readings by means of the calibration curve W in Fig. 23(*a*) furnishes the value for the pressure on the cell and not the average value q of the pressure on the wall. Since the evaluation of the cell readings by means of curve W (Fig. 23(*a*)) leads to a K-value of approximately 0.4, the real value of K should be expected to be between 0.3 and 0.4.

The uncertainties involved in the interpretation of Mr. Duke's cell readings could have been avoided by reinforcing the stressmeter plates by means of closely-spaced steel ribs to a distance of several feet above and below each cell. Such reinforcement would have eliminated the yield of the stressmeter plate above and below each cell, and, as a consequence, an evaluation of the strainmeter readings by means of curve W in Fig. 23(*a*) would have furnished reasonably accurate results.

On the basis of his value $K = 0.7$, the author computed the deflections of the bulkhead and found that the computed deflections are approximately equal to the measured ones. Therefore, he concluded that the value $K = 0.7$ must approximate the real one. The computation was based on the assumption that the distribution of the earth pressure on the two sides of the bulkhead is that shown in Fig. 14 However, it is no exaggeration to say that the assumed pressure distributions are fictitious. The real distribution of the passive earth pressure over the outer face of the buried parts of the sheet pile depends entirely on the flexural rigidity of the piles and the compressibility of the adjacent soil; consequently the deflection of the piles at a given intensity and distribution of the active earth pressure can vary between wide limits. This has been demonstrated by a long series of very instructive bulkhead tests performed by P. W. Rowe.[18,35] These tests disclosed the fact that if the flexural rigidity of the sheet piles is increased, the maximum bending moment in the piles may increase by as much as 300% with no alteration in the active earth pressure on the inner face of the bulkhead. The increase of the maximum bending moment is almost exclusively the result of the influence of the flexural rigidity on the distribution of the passive earth pressure over the outer face of the buried part of the sheet piles, and on the location of the center of pressure. This fact invalidates the results of Mr. Duke's interpretation of the deflection data. The data are theoretically compatible with any value of K between 0.3 and 0.7, depending on the assumptions made regarding the distribution of the passive earth pressure.

The preceding discussion leads to the following conclusions: (*a*) The pressure cells used for determining the soil pressure on the contact face of the cells had some defects that precluded reliable calibration; (*b*) because of the flexibility of the stressmeter plates, the unit pressure on the contact face of the cells was considerably greater than the average unit pressure on the bulkhead at the elevation of the cells and this fact has not been considered; (*c*) the cells in contact with rock fill were subject to eccentric loading and, as a consequence, there is no definite relationship between the pressure on these cells and the cell readings; and (*d*) the determination of the K-value of the backfill on the basis of the results of the deflection observations on the bulkhead involves serious errors caused by the uncertainties associated with the assumptions concerning the distribution of the passive earth pressure over the face in contact with rock fill.

Because of these facts, the observational data contained in the paper do not permit a reliable evaluation of the K-value of the backfill of the Long Beach bulkhead. They do not, therefore, justify the conclusion that this value is greater than 0.4, which is the upper limiting value compatible with the present (1953) knowledge of the mechanical properties of sand. The value of K for this backfill may even be smaller than 0.4.

Piezometric Data.—Table 5 contains the results of piezometric readings. A digest of the data contained in the table may produce very instructive results. For example, Fig. 25(*a*) represents the hydraulic conditions that prevailed in the fill on June 11, 1949, at 5:30 a.m. The tide was rising and the tide level was still very low (+0.9 ft). The hydrostatic pressures in the pore water of the

[35] Discussion of "Anchored Sheet-Pile Walls," by Peter W. Rowe, *Proceedings*, Inst. of C. E., Vol. I, Pt. 1, September, 1952, pp. 616–647.

fill are determined by the readings on the piezometric tubes H-1 to H-6. At the tip of every piezometric tube the hydraulic head is equal to the difference between the elevation of the water in the tube and the elevation of the tip of the tube. The location of the tips of the tubes is shown in Fig. 25(*b*). In Fig. 25(*a*) the readings on the gages H-1 to H-4 are indicated by small circles and those on gage H-6 by a cross. The increase of the hydraluic head with depth is shown in Fig. 25(*a*) by the broken line A B C. The position of the section BC of the line with reference to the vertical axis shows that the pore water in the rock fill behind the bulkhead communicated almost freely with the open water

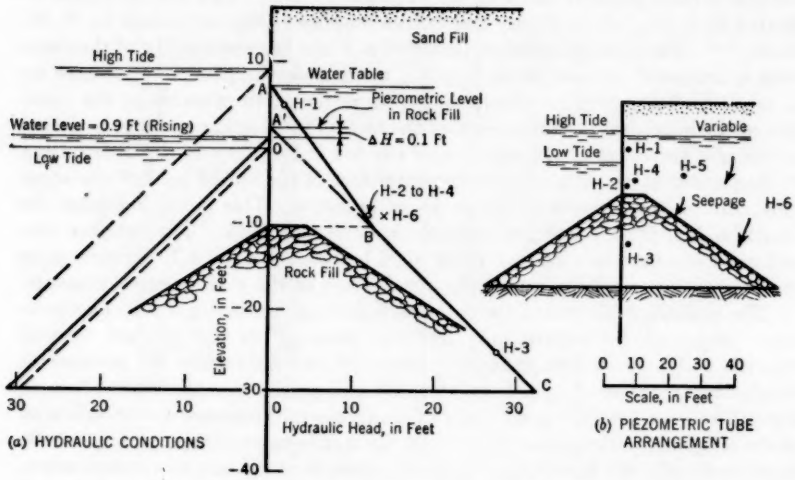

FIG. 25.—HYDRAULIC PRESSURE CONDITIONS IN THE FILL FORMING THE PIER, ON JUNE 11, 1949, AT 4:30 A.M.

because the piezometric head is only approximately 0.1 ft above the free water level (Fig. 25(*a*)). In fact, only twice during the entire period of observation of almost five months did the difference ΔH exceed a value of 1 ft, although the tidal range was nearly 10 ft. The position of the upper part AB of the pressure line with reference to the upward continuation A'B of BC shows that the sand fill was completely consolidated and that it drained during low tide in a downward direction toward the rock fill. In Fig. 25(*b*) the direction of the flow of seepage is indicated by arrows. In order to get a clear conception of this process it would have been necessary to make at least one piezometric reading per hour over a period of twenty-four hours, supplemented by simultaneous determination of the elevation of the free water level in the harbor, on two or three different days. Considering the importance of this information and its bearing on the earth pressure conditions, it is suggested that such observations be made. The results might be published by the author in the closure to the discussion, together with information concerning the tidal characteristics of the harbor. It also would be useful to the readers to know the inner diameter of the piezometric tubes, because this diameter and the coefficient of permeability of the fill

determine the time lag between the change in the pore-water pressure and the corresponding change of the water level in the tube.

Performance of Tie Rods.—The bottom set of diagrams in Fig. 12 shows that the major part of the fill above the tie-rod level is carried by the tie rods and that it acts on the tie rods as an ice load does on an electric conduit suspended between transmission towers. This loading condition invalidates the design assumptions for the tie rods and is inevitable if the tie rods are buried in a backfill that rests on a subsoil containing highly compressible layers. Elimination of this condition by installing the tie rods on the bottoms of conduits allows the fill to subside and the conduits to settle without affecting the rods.

Soil Test Data.—Under the heading, "Structure and Materials," Mr. Duke describes the laboratory tests that have been performed on many samples from twenty borings. Unfortunately, the most important property of the fill material—its relative density—has not received any consideration. If the fill material is dense, its angle of internal friction ϕ is great and a slight yield of the lateral support is sufficient to reduce the lateral pressure exerted by the fill to its minimum value. However, if the fill is loose, its ϕ-value is small and the lateral pressure decreases at a slow rate, while the bulkhead yields. The ϕ-values of 38° and 37° appearing in Table 1 for the upper strata *a* and *b* of the hydraulic fill are ϕ-values for a sand in a dense state, but it is unlikely that the fill really is dense. Considering the large quantities of time and money that have been invested in the study, it would certainly be justified to make one more exploratory boring through the fill material, combined with standard penetration tests, to secure reasonably reliable information concerning the relative density of the sand fill. Accurate information conceivably could be obtained by laboratory tests on undisturbed samples, but the recovery of such samples from sand strata is impracticable unless elaborate and expensive precautions are taken, such as chemical solidification or the freezing of a plug at the lower end of the sample.

Extensive testing has been performed in the laboratory, and valuable information can be extracted from these data by constructing soil profiles showing curves of equal effective grain size for both the fill and the natural ground. There is an urgent need for reliable information concerning the pattern of stratification of both hydraulic fills and natural sand deposits. Profiles of this kind would be a valuable and appreciated addition if added to the closure. If the borings are combined with standard penetration tests, the soil profiles should be supplemented by others showing curves of equal numbers of blows, which furnish approximate information concerning the variation of the relative density within the strata. The opportunity for securing such profiles should never be missed.

Uncertainties Involved in Bulkhead Design.—According to a statement in the "Introduction," anchored flexible bulkheads are not yet amenable to accurate analysis "*** because of insufficient understanding of the performance of the prototype system." In the writer's opinion the reason is a different one. Knowledge of the performance of bulkheads in general has already advanced to a state which leaves little to be desired (1953). However, as knowledge of the subject increased, engineers realized that some of the most

important factors on which the performance depends—such as the pattern of the variation of the compressibility of the soil in contact with the buried part of the sheet piles—cannot reliably be determined by any practicable means. Hence, the major uncertainties associated with bulkhead design are not caused by broad gaps in the knowledge of bulkhead performance. They result from the fact that the time and money that reasonably can be spent on subsoil exploration are limited, and this condition will never cease, irrespective of what engineers may learn.

Practical Value of the Investigation.—In spite of his objections to some of Mr. Duke's statements, the writer feels that the results of the investigation fully justify the work and expenditures involved. These results demonstrated emphatically the effect of a subsidence of the fill on the performance of the tie rods. They showed that the recession of the tide may be associated with a flow of seepage through the fill material involving a temporary increase of the effective weight of the fill. They also call attention to the inevitable uncertainties involved in bulkhead design. Hence, investigations of this kind certainly should be encouraged, whatever their shortcomings may be, because their results are the only source of information concerning the inevitable errors involved in the application of theoretical procedures of any kind to bulkhead design. It is always possible somehow to "extract the gold from the gravel." The Long Beach Harbor Department deserves the gratitude of the profession for having sponsored and financed the investigation.

D. P. KRYNINE,[36] M. ASCE.—Members of the engineering profession are always interested in the proper approach to the design and construction of water-front structures such as bulkheads, and important results of laboratory work on bulkhead models have been published. Although these findings were widely publicized, practical engineers are doubtful as to the unquestionable validity of this model research unless the results of such research are checked by observations on the prototypes under actual in-service conditions. It is difficult to reproduce storms and tides, or even hydraulic-fill construction. Wave action is a common phenomenon in the ocean and is apparently responsible for some of the features involved in bulkhead performance, which escape or are misinterpreted by a laboratory investigator.

The paper tends to fill the existing gap, but it was impossible to cover the whole field and to answer all pertinent questions. The writer was present at the initial stages of Mr. Duke's work. There was simplicity and efficiency in the instrumentation, combined with energy and a wide outlook, for the performance of this pioneer work. The writer has no doubt as to the correctness of the reported data, but merely wishes to revise the general opinion on the subject matter in the light of Mr. Duke's findings.

Action of the Dike.—The horizontal pressures exerted by the outer part of the dike on the sheet piling (shown in Fig. 6 and termed passive resistance) equal the horizontal stress components at the outside face of the sheet piling. The horizontal pressures exerted by the inner part of the dike on the sheet piling (active pressure) equal the horizontal stress components at the inside

[36] Cons. Engr., San Francisco, Calif.

face of the sheet piling. If the sheet piling were a thin, stressless membrane, horizontal pressures on both sides of the membrane would be equal. In this case, the horizontal pressure diagrams on both faces of the sheet piling would be identical. Because of the relative stiffness of the sheet piling, the numerical values of the abscissas of these diagrams would be different on both faces of the sheet piling.

As a rough approximation, assuming that the diagrams in question are identical, and using the outside horizontal pressures from Fig. 12 and Table 3 for plotting the inside horizontal pressures, the diagrams for active pressures

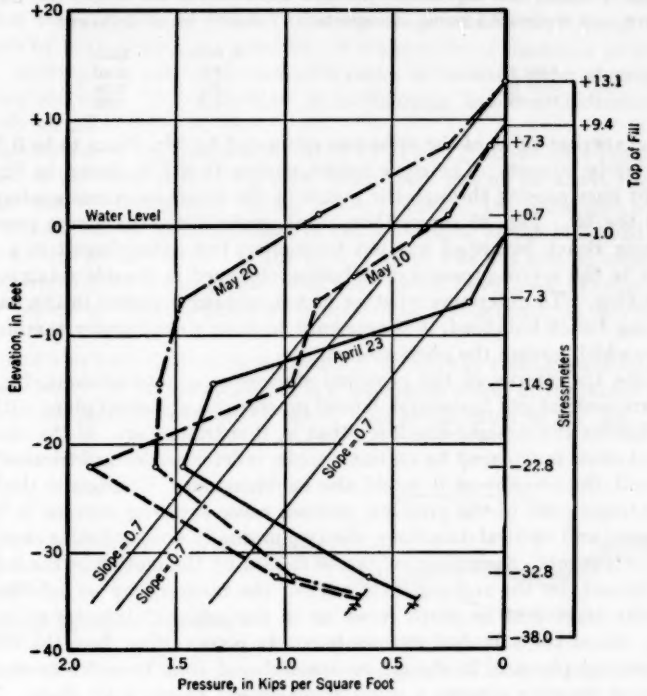

Fig. 26.—Active-Pressure Diagrams at Station 27 + 30

acting on the bulkhead may be plotted, as in Fig. 26. In using this figure, it should be remembered that all abscissas above El.-14.9 are measured values, whereas those below El.-14.9 are assumed values which are fair approximations.

Field Active-Pressure Diagrams.—In Fig. 26, three field active-pressure diagrams are superimposed. They were drawn on the basis of research data obtained during the filling operations.

These three diagrams have been chosen at random from Fig. 12, and they can be considered as representative samples of the active-pressure distribution before changes caused by the settlement of the fill occurred on June 11. Con-

sidering the upper part of Fig. 26 (above El.-14.9), the writer believes that the first sentence under the heading "Summary of Test Results," should read as follows:

> During the filling operation, the pressure of fill against the bulkhead in the upper part of the latter increased as the weight of the overlying fill was increasing.

It should be noted that, at the top of the April 23 curve in Fig. 26, the ratio between the horizontal pressure and the weight of the overlying fill is smaller than in the underlying layers located above El.-14.9. In fact, this ratio (the distance between the top of the fill and the stressmeter where the horizontal pressure was measured being designated h) would be as follows:

Date	h, in feet	Ratio
April 23	6.3	0.50
May 10	8.7	0.54
May 20	5.8	0.82

The average value of the ratio was estimated by Mr. Duke to be 0.7, which is obviously correct. The slope corresponding to 0.7 is shown in Fig. 26 as straight lines passing through the points of the diagrams corresponding to the top of the fill. Fig. 26 shows that, in all probability, the active pressure on the Long Beach bulkhead was not triangular, but redistributed in a manner similar to the active-pressure distribution observed in flexible retaining walls.

Arching.—To determine whether or not arching occurred in the backfill of the Long Beach bulkhead, it is necessary to have a clear understanding of the factors which govern the phenomenon.

If the trajectories of the principal stresses in a homogeneous, fragmental mass are vertical and horizontal, lateral pressure on a vertical plane within that mass follows the straight-line law—that is, it is triangular. If the imaginary, vertical plane is replaced by an immovable, indeformable, and frictionless vertical wall, the pressure on it would also be triangular. Changes in the position of the trajectories of the principal stresses cause shearing stresses in both the horizontal and vertical directions, the two mutually perpendicular shears being equal at a point. Such changes can be caused by the motion of the mass, or a part thereof, by the removal of a part of the mass, or by an addition to it. Irregular placing of an earth mass, as in the case of dumping or hydraulic filling, causes the principal stresses to act in planes other than the horizontal and vertical planes. It should be remembered that in order to mobilize a system of shearing stresses a small displacement should take place. Thus, if a homogeneous, fragmental mass is bounded by an immovable, vertical plane which is prevented from sufficient bending by its rigidity, the trajectories of the principal stresses in this mass would remain horizontal and vertical, and the active-pressure distribution would be triangular.

Shearing stresses of a certain magnitude, developed horizontally and vertically on the trajectories of principal stresses of a homogeneous fragmental mass, cause a shifting of these trajectories. At all points through which the new trajectories pass, the state of stress changes, since all shears at these points vanish. The adjacent particles are thus given an opportunity to press against one other without any disturbance or tendency toward lateral displacement

caused by shears. If pressed from both ends, this trajectory may stretch or arch; hence, the most probable arching curves in a sand mass are the trajectories of the principal stresses. Stretching or arching along a trajectory of principal stresses can occur only if there are two relatively immovable points at the ends of that trajectory. Presumably, the arch thus formed should also satisfy the requirements of stability—and if one of the abutments moves, the stretching loosens, and the arch breaks. Since major principal stresses generally begin at the free surface of the earth mass (the backfill, in the case of a bulkhead), arching along the trajectory of the major principal stress may be expected only occasionally. The arches into which the trajectories of the minor principal stresses are transformed are loaded (not necessarily uniformly) along their total length by normal loads provided by the trajectories of the major principal stresses. Arching is a particular case of transfer or redistribution of pressure by shearing stresses. This difference in terminology has been indicated by Mr. Tschebotarioff.

An examination of the diagram for April 12 (Fig. 12) shows that the stress distribution in the outer part of the dike, when there was no fill above the top of the dike, was not triangular. In the writer's opinion, the outer part of the dike was arched, one of the abutments of the arch being at the bulkhead and the other at the ground level. If the ground level was too soft to offer the resistance required for arching, this resistance would have been provided by the lower layers of diabase particles pressed into the ground, as shown in Fig. 27.

Engineers who have dealt with settlements of highway embankments know that the settlement of an embankment is a maximum either at its center line, or at two points on both sides of the center line, as indicated in Fig. 28. The

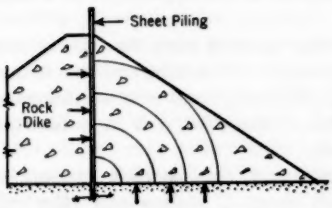

FIG. 27.—ARCHING IN THE DIKE

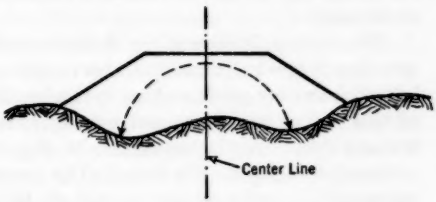

FIG. 28.—SETTLEMENT OF A ROADWAY EMBANKMENT

latter case is a result of an arching with redistribution of the pressure on the ground and a relief of pressure at the center. This analogy is mentioned in order to support the opinion concerning the arching in the Long Beach dike. Arching similar to that shown in Fig. 28 requires a favorable background, such as (1) firm support at two places on both sides of the center line and (2) earthwork by dumping rather than by compacting regularly-placed thin lifts.

A movement of the wall away from the earth mass it supports causes horizontal arching in the latter, as first shown by Mr. Terzaghi. In fact, one abutment of the arch is on the wall, and the other is on the potential shearing surface which is a physical reality such as the wall itself. It is clear that terms such as vertical or horizontal arching, often used in describing certain

arching phenomena, are inadequate and should be eliminated from soil mechanics nomenclature. The arching shown in Fig. 27, which took place along the trajectories of the minor principal stresses, was neither vertical nor horizontal.

Arching of the Backfill of the Long Beach Bulkhead.—It is difficult to answer categorically whether or not there was arching of the backfill behind the Long Beach bulkhead. Possibilities for arching did exist there. For example, a trajectory of the minor principal stress, with one abutment at the tie rods and the other on or next to the heel of the dike, was in a condition to cause arching under the action of a convenient system of forces. Such an arching would have decreased the bending moment in the span below the tie rods. In work similar to hydraulic filling, the trajectories of principal stresses shift their location continually, and arches are formed and broken. In this connection, irregular, so-called "erratic" readings may be obtained. An example is the excessively protruding abscissa of the diagram for April 23 in Fig. 26. These erratic readings are quite correct if the instrumentation works properly; however, the pressures are erratic.

In the case of the Long Beach bulkhead, the fact that the active-pressure distribution on the bulkhead was not triangular, but was redistributed by shearing stresses was important. Because of the physical nature of shearing stresses, they cannot create pressure, but can only redistribute or transfer it; that is, they are self-balanced. In the upper part of the curves of Fig. 26 there is an excess of pressure (in comparison with the triangular distribution) which is being caused by horizontal shears acting toward the bulkhead. Since these shears should be balanced, the logical conclusion is that, at the lower part of the bulkhead, there should be shearing stresses acting in the opposite direction—that is, decreasing the pressures. This means that the general shape of the curves in Fig. 26 can be justified without the use of the hypothesis of a thin membrane.

Translatory Motion of the Bulkhead.—In the general case, backfilling not only bends the bulkhead but also causes a translatory motion, and this translation alone may cause arching in the backfill. In the present case the influence of this factor was decreased by the presence of the dike; yet in bulkheads without dikes, this factor cannot be neglected.

Overloading of the Tie Rods.—The overloading of the tie rods cannot be explained by arching from tie rod to tie rod, since, in such a case, the soil material above the tie rods would not move downward, being supported by the arches. This would cause the material below the tie-rod level to settle away from the tie rods; but such a phenomenon did not take place.

A probable explanation consists in the formation of shearing surfaces above each tie rod. Such shearing surfaces would be closed surfaces, and if there is not enough vertical distance for their development, they would intersect the surface of the backfill. Soil material enclosed by these shearing surfaces was supported by the tie rods and made them sag. In addition, there was drag (negative pressure) on the shearing surfaces.

In connection with the overloading of the tie rods, there should have been back motion of the bulkhead. The upper abutment of the arches, shown in Fig. 27, then moved toward the inside of the backfill, and the arches loosened

and broke. This is represented by the diagrams of August 5 and the following days (Fig. 12).

Suggestions for Future Research.—Mr. Duke is to be commended for presenting suggestions to future field investigators of anchored bulkheads. The writer wishes to add to item 3 of these suggestions that, if the soil material at a given project is more or less uniform, moisture content at different elevations could be calibrated against density and shearing strength. Thus, in addition to a relatively limited number of undisturbed sample testings, a large number of moisture-content determinations could be made, thus giving a number of approximate density and shearing-strength values.

TRENT R. DAMES,[37] M. ASCE, AND DAVID C. LIU.[38]—New techniques in field instrumentation and new information on bulkhead behavior are presented by the author. The paper represents the first important published result of a careful study of an anchored bulkhead. He is to be complimented for successfully accomplishing this study in such a careful manner. The engineering profession also owes much to the Long Beach Harbor Department for cooperating in this research. The writers hope that similar studies will be made often, in order to furnish field evidence for comparison with classical methods of analysis and laboratory experimental data.

The writers will discuss (*a*) the design of the Long Beach bulkhead, (*b*) the reported pressure observations, (*c*) the author's suggestions concerning future research, and (*d*) possible additional study of the Long Beach bulkhead.

(*a*) The use of a granular dike to reduce the pressure from the more fluid hydraulic fill at the lower elevations is commendable. Having the rock dike on both sides of the bulkhead not only provides the effect of a reduction of the pressure acting on the inside of the dike but also a considerable increase in the resisting pressure on the outside. Using this construction, it might have been possible to have reduced the length of the sheet piling from that shown in Fig. 2.

The anchor piles (with 40-ft penetration into natural sand and 3-ft spacing) have much batter and therefore are comparatively unyielding. Even with the high stress of 32,000 lb per sq in. observed in the tie rods at Station 27+30, the sheet piling could deflect outward by only approximately $\frac{3}{4}$ in. relative to the anchor cap. This deflection, together with the small movement of the relatively unyielding anchor cap, is quite insignificant in terms of soil deformation behind bulkheads; and, therefore, the soil pressure near the wale should be close to the "at-rest" value.

It is conceivable that a more flexible system of anchorage, together with the use of ties protected from sagging caused by settlement of fill, would have resulted in lower tie-rod tensions.

(*b*) The information presented by the author concerning observed soil pressures is of great interest and importance because the observed values exceed normal assumptions by more than 100%.

To explain this difference, it is necessary to consider the pressure-deformation relationship of soils. It is not possible to establish quantitative relationships for any soil without extensive experimentation, but it is safe to say that,

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[38] Eng.-Analyst, Dames and Moore, New York, N. Y.

when a bulkhead moves away from the soil, the pressure against the bulkhead will be reduced. It is possible that the high pressures reported in this paper were caused, at least in part, by the lack of appreciable outward deformation of the wale—it is doubtful whether the wale actually moved outward after the completion of filling early in June. The tie-rod tension increased approximately 25% between June and October at Station 27 + 30, indicating an elongation of approximately $\frac{1}{8}$ in. In the meantime, the fill settled and caused the tie rod to sag. This sag would tend to pull the wale and the anchor cap together, probably by an amount greater than the elongation of the tie rods. Thus, there was never sufficient outward movement of the bulkhead to reduce the lateral soil pressures to values normally assumed.

(*c*) The author's nine suggestions under the heading, "Suggestions for Future Bulkhead Field Studies," are of particular value to anyone who may perform field studies of earth structures. The experiences in instrumentation reported in this paper should prove invaluable as background for planning future field studies. It might be desirable, relative to suggestions 6, to install gages measuring strain in a vertical direction on both sides of a bulkhead in order to detect any appreciable friction on the inside. The gages could be protected by pieces of channel sections covering the gages and welded to the bulkhead on one flange immediately above the gage.

(*d*) At present (1953), it is understood that the Long Beach bulkhead will be extended upward and that the stresses in the tie rods will be relieved by excavation of fill from around the rods and loosening of the turnbuckles. In an operation of this type on any similar bulkhead, the change in tie-rod tension accompanying the loosening of the turnbuckles might be determined by pulling the tie rod on the outboard side of the bulkhead, holding the wall, and finding the pull on the tie rod necessary to separate the rod from the wall. Strain gages can be attached to tie rods with known stresses, and the subsequent changes in stresses determined. To avoid local horizontal redistribution of pressure, at least three adjacent tie rods should be released at the same time; observations should be made on all three. Perhaps observations of this nature would be of value in determining a proper range of elongations for all the ties. It might be well to protect the gaged rods, and possibly all the rods, against subsequent fill settlement by enclosing them in boxes.

Roy W. Carlson,[39] A.M. ASCE.—Some caution is suggested in accepting the author's finding that the horizontal pressure against the bulkhead is approximately 70% of the vertical load. The lateral pressures were determined by use of stressmeters and checked by tie-rod tensions measured by strainmeters. The certainty of the results is doubtful because of the peculiar characteristics of the particular stressmeters used. Furthermore, the check by measurement of the tie-rod tensions was not exact; it was dependent on assumptions concerning the pressure distribution at lower levels. The exact method of determining pressures from the stressmeters must therefore be investigated.

The calibration curves for one stressmeter (Fig. 9) shows that the response was nonlinear and was markedly different for water pressures and soil pressures.

[39] Cons. Engr., Berkeley, Calif.

After the Long Beach study was completed, the writer secured several similar stressmeters. These meters were found to have a high and nonlinear compressibility, indicating the presence of air entrapped with the mercury. This indication was confirmed by the fact that the response became linear when the meters were refilled with mercury. High compressibility would allow a large part of the load to be carried by the metal rim—especially under types of

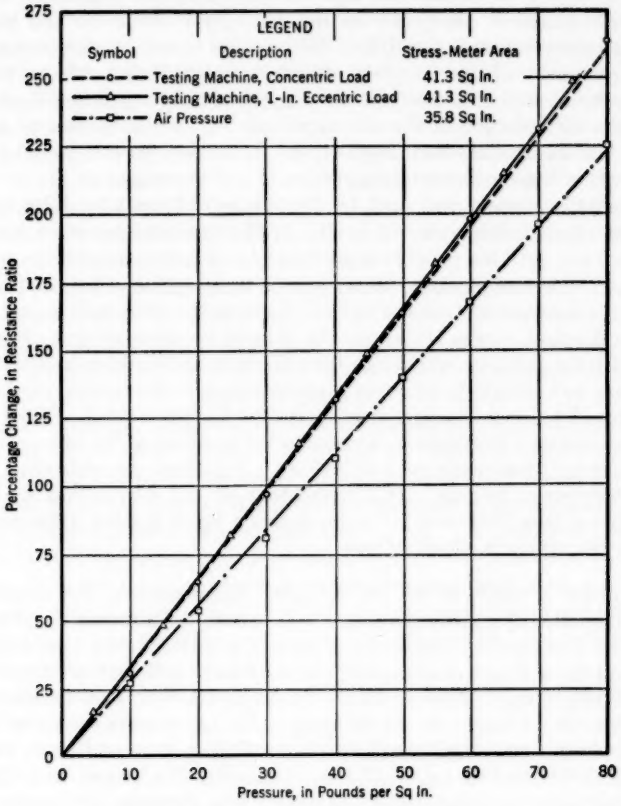

FIG. 29.—STRESSMETER CALIBRATION

loading where the plates of the stressmeter were not free to "dish." The response should be greatest with water loading, since the plates could dish and most of the pressure would be applied to the mercury film. At low pressures the response is only partial with soil pressure loading when the rim carries much of the load; the response is more nearly normal at high pressures, when the entrapped air is highly compressed. The calibration curves of Fig. 9 are in accordance with such reasoning.

If the stressmeters have these characteristics, a modified manner of using the calibration curves is necessary. Where both water pressure and soil pressure are acting, it is not correct to use both the calibration curves for water and soil, respectively, from the origin. A more correct procedure would be as follows: Since there was an initial water pressure, the calibration curve for water pressure should be used to determine what part of the observed change in resistance ratio of the stressmeter was caused by the water pressure. Following this determination, the point on the calibration curve for soil pressure having this same pressure is noted, and the intercept of soil pressure corresponding to the remainder of the change in resistance ratio is found. Thus, the soil-pressure calibration is used not from the origin but from a point corresponding to the initial compression of the entrapped air. It is not correct to assume that, after the water pressure is applied, the calibration for soil pressure is the same as if there were no initial compression of the entrapped air.

Although the stressmeters used by Mr. Duke did not all exhibit the unsatisfactory characteristics shown in Fig. 9, the indicated pressure would be considerably less with the procedure previously outlined than with the author's procedure. Therefore, it would be of interest to determine what the pressures were—on the assumption that the writer's views of the stressmeters are correct.

The calibration curves shown in Fig. 9 may be misleading to those unfamiliar with the progress which had been made in stressmeter development.[40] Stressmeters now (1953) have linear calibrations, and the results obtained are the same for all types of loading—even eccentric (within limits). When water pressure surrounds a stressmeter, account must be taken of the area subject to pressure, just as is done with sylphon bellows. Fig. 29 shows calibration curves for various types of loading. It is noted that all the calibrations are linear, and that the calibration curve for water pressure has a slightly different slope, because of the different effective area.

C. MARTIN DUKE,[41] A.M. ASCE.—The discussions of the paper have added to its value by comments on the test results, by alternative interpretations of the findings, by suggestions regarding further field and analytical studies, and by a direct challenge of the reported coefficient of lateral earth pressure during filling. Messrs. Lake, Terzaghi, Carlson, and Tschebotarioff have studied the validity of the stressmeter measurements. Nearly all the discussers have commented regarding the coefficient of lateral earth pressure and the loading of the tie rods. Messrs. Baumann, Packshaw, Terzaghi, and Krynine discussed the behavior of the dike. Mr. Terzaghi interpreted some of the pore pressure measurements. Several of the discussers made suggestions for additional field research on anchored bulkheads. Messrs. Boyer, Way, and Terzaghi commented on the philosophy of the field study approach. Mr. Packshaw and Messrs. Dames and Liu gave special attention to the subject of design of flexible bulkheads as influenced by the Pier C test findings.

Validity of Stressmeter Measurements.—The values of soil pressure as computed from the readings of the stressmeters were questioned on the bases that

[40] "Development of a Device for the Direct Measurement of Compressive Stress," by R. W. Carlson and David Pirtz, *Journal*, A. C. I., November, 1952.

[41] Associate Prof. of Eng., Univ. of California, Los Angeles, Calif.

(1) the diameter of the stressmeter contact face was too small, and not enough of these instruments were used; (2) the method by which the stressmeter calibrations were utilized was incorrect; and (3) the method of mounting the stressmeters used was questionable.

The number of stressmeters used was limited by economic circumstances. In planning the test program it was therefore hoped to obtain an over-all indirect determination of soil pressure by measuring tie-rod tensions. The diameter of the individual cells was only about 7½ in., which as stated in the paper is certainly too small for reliable quantitative measurements in the dike. Considering the nature of all soils, especially those deposited in full-scale hydraulic filling operations, it certainly would have been desirable to have had a larger contact area, even in the sand. Mr. Tschebotarioff has suggested a device (Fig. 21) whereby this might have been accomplished. An arrangement similar to that shown in Fig. 21 was used successfully by Mr. Terzaghi in measurements on the Chicago subway.[42] Mr. Lake indicates the apparent variation of soil pressure horizontally along the bulkhead as indicated by the variation in measurements of tie-rod tension.

Mr. Terzaghi has discussed the reliability of the particular stressmeters used, of their calibration, and of the way in which they were mounted on the bulkhead. Mr. Carlson's discussion answers some of the questions raised by Mr. Terzaghi and also gives an opinion regarding the proper use of the particular soil pressure and water pressure calibration curves obtained. It is clear that the mercury chamber of the stressmeters contained air, which accounts for the nonlinearity of the two curves shown in Fig. 9. Presence of this nonlinearity was most unfortunate; the writer is happy for the sake of future users of the stressmeters that Mr. Carlson has devised a method[40] of permanently eliminating all air from the mercury chamber (see Fig. 29). The Pier C stressmeters were fabricated by an inexperienced manufacturer. If it is to be emphasized, however, that the actual calibration curves obtained, both with soil and with water, were reproducible to a fully satisfactory accuracy, the particular curves used for the respective stressmeters can be expected to give reliable values of either water pressure or soil pressure acting alone.

Fig. 30 gives the soil pressure calibration curves of all stressmeters for which results are presented. The soil pressure calibrations were made in the laboratory as shown in Fig. 29, except that the stressmeters were bedded on a firm, stiff rubber pad 1 in. thick and of a hardness of from 60 to 70 Shore, and the load was centered on the meters. Choice of the rubber bedding was based on a series of experiments conducted at the University of California at Berkeley in 1944 and subsequently substantiated by field measurements at an airport pavement test program[4] where satisfactory checks were obtained between the readings of stressmeters beneath the pavement and loads applied on the surface of the pavement.

The fact that the calibration curves for both soil pressure and water pressure for the Pier C stressmeters were nonlinear and were different from one another

[42] "Liner-Plate Tunnels on the Chicago (Ill.) Subway," by Karl Terzaghi, *Transactions*, ASCE, Vol. 108, 1943, p. 991.

gave the writer considerable concern during the test program and the subsequent analysis of data. To solve the problem of finding the soil pressure from a stressmeter reading by utilizing a known water pressure and the two nonlinear calibration curves is not easy. Messrs. Terzaghi and Carlson very properly call attention to this characteristic of the instruments and challenge

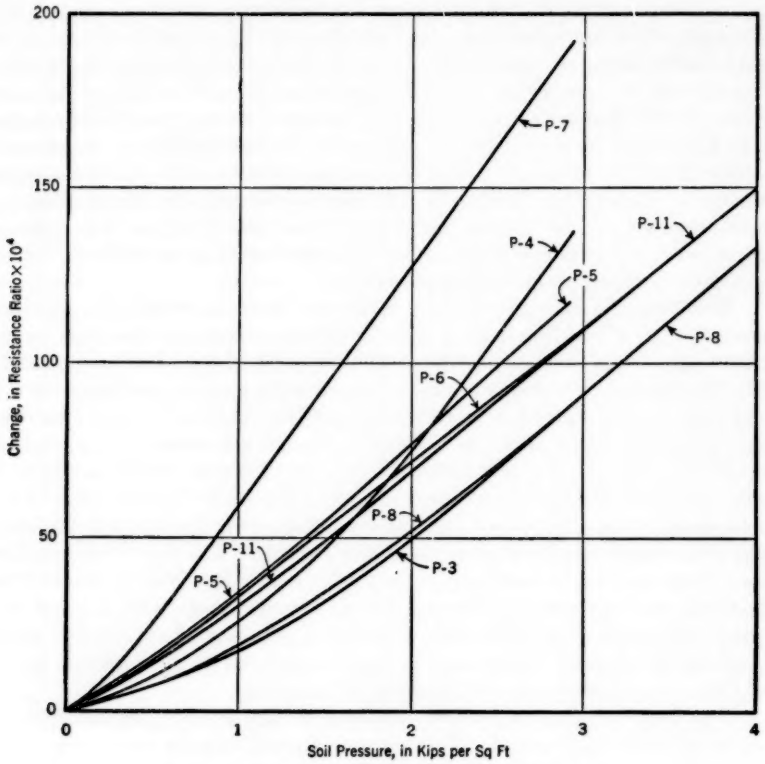

FIG. 30.—SOIL-PRESSURE CALIBRATION OF STRESSMETERS

the writer's use of the two curves. Mr. Carlson explains clearly why the nonlinearities exist. He also explains (in terms of the presence of air in the mercury layer) why the water-pressure calibration curve should lie above the soil pressure calibration curve (Fig. 9). The procedure adopted in the paper was, in effect, to use the parts of the two curves near the origin. The writer has discussed this subject with Messrs. Carlson and Terzaghi and agrees with Mr. Carlson that a better procedure would have been as follows: The calibration curve for water pressure should be used to determine what part of the observed change in the resistance ratio resulted from water pressure; the reader should then move vertically to a point on the calibration curve for soil pressure and

use the remaining amount of resistance-ratio change to move to a higher point on the soil pressure curve; the horizontal intercept between the latter two points should be taken as the recorded soil pressure. This procedure is preferred to that (suggested by Mr. Terzaghi) of using the water pressure calibration curve for both water and soil because (as explained by Mr. Carlson) the shapes of the two calibration curves theoretically should be different. The effect of the revised (Carlson) procedure is to reduce appreciably the values of soil pressure indicated by the stressmeters having the more nonlinear soil pressure calibration curves. The nature and extent of this reduction will be considered subsequently. The writer agrees with Mr. Terzaghi that the stressmeter mounting plates were too flexible at points away from the meters and that this tends to cause recorded pressures to be too great (Fig. 24).

Behavior of Dike.—The writer agrees with Messrs. Baumann, Terzaghi, and Krynine that the lack of adequate data on the behavior of the bulkhead below the top of the dike is regrettable. The difficulty of obtaining quantitative pressure measurements in the dike has been emphasized by several of the discussers. Probably some qualitative information can be gleaned from the Pier C measurements. Two stressmeters in addition to P-8 (Fig. 6) were placed in the dike on the inner side of the bulkhead, but these were damaged during the placing of the fill and no measurements were made with them.

Messrs. Terzaghi and Packshaw have commented on the assumptions regarding pressure distribution between the dike and the bulkhead, on which the partial check between tie-rod tension and soil pressure measurements was based. There are several possible assumptions about the dike pressure distribution; the check of tie-rod tension against soil pressure is therefore only partial. The writer does not in any sense advance the procedures he used in producing Fig. 14 as desirable design practice.

Mr. Krynine has suggested an ingenious scheme for using the measured pressures in the outer dike to approximate those in the inner dike, on the assumption that the stiffness of the sheet-pile wall is negligible. Qualitatively, at least, this gives a result (Fig. 26) consistent with the assumption of Fig. 14. The concept of membrane action would be most valid at places where the shear in the bulkhead is small (see Fig. 15). The upper two lines labeled "Slope = 0.7" in Fig. 26 should be broken to show a flatter slope at the groundwater level.

Messrs. Tschebotarioff and Krynine have given considerable attention to the subject of "arching." The writer agrees with Mr. Tschebotarioff's deduction from the test data that there was no evidence of arching caused by deflection of the bulkhead. No observable change in deflection occurred at any point on the bulkhead during the pressure redistribution of Fig. 12. Mr. Krynine's general discussion of the subject of arching with application to the situation at the Pier C bulkhead (Figs. 27 and 28) should be read as an aid to understanding the basic mechanism of the arching phenomenon. In the opinion of the writer, the shear-stress concept is the only rational approach that can be made to this complex phenomenon.

Messrs. Dames and Liu comment on the function of the dike and rightly explain that the designer should take full advantage of its structural characteristics.

The silty clay found in borings A-1 and A-2 and shown in Table 1 was laid down during the filling process, which postdated the placing of the rock dike. Except for this fact, Mr. Lake's suggestion for a possible rotation with respect to the tie-rod level of the lower part of the bulkhead, including the dike, would be feasible.

Earth Pressure Against Bulkhead During Filling.—Most discussers noted the magnitude and distribution of lateral earth pressure against the bulkhead during the filling operation. Mr. Terzaghi challenged the validity of the reported coefficient of lateral earth pressure (K equals 0.7) on the bases of the stressmeter calibration and reliability and the experiences of previous investigators with determinations of earth pressure at rest. Messrs. Tschebotarioff, Lake, and Packshaw, and Messrs. Dames and Liu observed the unexpectedly high value of this coefficient and offered possible explanations for it. Messrs. Krynine and Tschebotarioff have correctly pointed out that the concept of a single-valued coefficient is fallacious. The value of K probably varies both with time and with elevation on the bulkhead.

Based on the preceding discussion of the stressmeter calibration curves, the writer has recomputed some of the soil pressures obtained with those stressmeters having the more nonlinear calibrations and has found that this appreciably reduces the coefficient of lateral pressure during filling. Such a reduction occurs because the more nonlinear the stressmeters used (P-3, P-4, P-8) all happened to be located where they had a strong influence on the computation of the value of K.

Mr. Tschebotarioff suggested that the high value of K reported for Pier C by the writer may have been caused by vibration accompanying wave action on the bulkhead. Mr. Lake suggested that the water-level differential between the inside and the outside of the bulkhead during three weeks of the filling operation (Table 3) could be responsible for a value of K that was higher than normal. The high inside water level could easily have caused large bulkhead deflections that could not subsequently reverse against the passive soil resistance. Mr. Packshaw suggests that the high value of K may result from arching of the soil between the forward-raking batter piles and the bulkhead wall during the filling operation. Messrs. Dames and Liu observe that the deflection of the bulkhead probably was not sufficient to cause appreciable reduction of the lateral earth pressures to less than the value of earth pressure at rest.

The writer's conclusion with regard to the lateral earth pressure, after study of the discussions and re-evaluation of the data, is that the coefficient of lateral earth pressure during filling was smaller than that reported in the paper. However, the errors and uncertainties discussed herein make it unwise to assign a numerical value to the coefficient.

Loading of Tie Rods.—In general, the discussers concurred with the writer's interpretation of the redistribution of soil pressure and the change in tie-rod tension following the completion of filling. That this tension change was

caused by the loading of the tie rods as a result of fill settlement is fairly well established. Practical remedies suggested included the placing of the tie rods inside concrete pipes and the provision of additional vertical supports along the length of the tie rods. Mr. Krynine rightly contends that complete soil arches in the architectural sense could not have developed between the tie rods, but rather that the drag of the tie rods with respect to the settling fill produces shear stresses without causing appreciable discontinuity in the soil.

Messrs. Tschebotarioff and Packshaw explain that the redistribution of pressure might logically be attributed to other causes of shear-stress distribution or arching. One of these causes is the presence of the rigidly supported anchor cap. Another is the presence of the closely spaced batter piles of the anchorage and the possibility of developing arch abutments on the batter piles and on the dike.

Mr. Boyer makes a strong contribution in computing the drag force that the fill would have to transmit to the tie rods in order to produce the tension increases observed subsequent to the completion of the filling operation. His computations indicate that a reasonable composite value for probable drag would be of the order of magnitude of 300 lb per lin ft of rod. He also shows that closer supports for the tie rods would greatly reduce the excessive tensions.

Pore Pressures.—Mr. Terzaghi reached essentially the same conclusions (Fig. 25) with regard to the effect of tide fluctuation on the pore pressures as were expressed by the writer under the heading, "Other Pressure Distributions." Direct use was made of the data of Table 5 in correcting stressmeter readings for water pressure and in computing those values of vertical soil pressure which required an allowance for seepage force.

The piezometer tubes were of $\frac{3}{4}$-in. inside diameter. Unfortunately, construction operations and the forces of nature have so damaged the instrumentation that it is no longer feasible to obtain additional data on pore pressures (as was suggested by Mr. Terzaghi). However, the data for August 5 and 6, Table 5, partly satisfy his request.

General Comments.—Most discussers made valuable suggestions for future field research on bulkheads. The comments of Mr. Packshaw and Messrs. Dames and Liu should be helpful to those interested in the design of bulkheads.

Although the quantity of information obtained from soil explorations and tests immensely exceeded that which the writer chose to include in the paper, the sixty-nine borings in the harbor area were too widely spaced to permit the construction of soil profiles as suggested by Mr. Terzaghi. Table 1 gives the general soil characteristics at Pier C on a quasi-statistical basis; the void ratio data permit computation of relative densities for the various strata. The actual void ratios (based on 3-in. Shelby tube push samples) are relatively low. Time did not permit the performance of Mr. Terzaghi's suggestion of making penetration tests in the fill.

Field studies in soil mechanics would benefit from a statistical approach. Although detailed study of specific earth structures of various types is necessary, much is lacking in such an approach that could be obtained by observations of only a few fundamental parameters at a large number of similar installations. For example, in a bulkhead study there might be measured at

twenty stations the tie-rod tension and the deflections at the wale, maximum bulge, and toe.

In closing, the writer desires to make clear certain limitations of the Pier C bulkhead tests.

1. The reported value of the coefficient of lateral pressure during the filling operation has not been determined with precision. The value reported in the paper, K equals 0.7, is too great. It is unlikely that this "coefficient" is constant; it probably varies both with time and with elevation on the bulkhead.
2. No quantitative conclusions are offered regarding the behavior of the dike or the passive resistance phenomena at the base of the bulkhead.
3. Important atypical features of the test were: The closely spaced batter piles; the silt layer at the bottom of the fill; the large differential between inside and outside water levels during an important part of the test; and the presence of the rock dike having a 3-in. maximum diameter.

The writer wishes to thank very sincerely the discussers, all of whom have made important additions to the value of the paper. Only a fraction of their many contributions could be considered in detail in this closure.

DISCUSSION OF LOAD TESTS OF SOILS IN THE MIAMI AREA
PROCEEDINGS-SEPARATE NO. 215

JAMES F. McNULTY, A.M. ASCE. — The engineer's job of choosing an allowable load on the basis of plate tests is certainly not an enviable one and can easily become, as the author so aptly puts it, a "hair-pulling ordeal." Before choosing a safety factor, he must make a decision upon the following points (1) what is "failure" for this type soil (2) how closely do areas tested represent true conditions (3) how closely does a 30-inch diameter plate represent a 10-foot square footing (4) if the soil is cohesive, what is the effect of consolidation and (5) how well were the tests run.

On Langley Air Force Base, plate tests were made for the National Advisory Committee for Aeronautics on the underlying stratum of saturated silty-sand (approximately 80% sand and 20% silt and clay). These tests indicated that, for both a 1-foot square plate and a 2-foot square plate, complete failure of load carrying ability was to be expected at approximately 8 tons per square foot; the larger plate, as expected, underwent greater deflection at less than failure load.

In addition, values of the internal angle of friction (ϕ) and cohesion (c) of the silty-sand were obtained from laboratory tri-axial tests. With these values, the theoretical failure for the plates could be calculated with the aid of available semi-empirical formulae.[1] Good agreement was obtained.

Since it is good policy to have borings taken on any project of magnitude, it is a simple procedure to have values of ϕ and c ascertained. Design could then proceed with the calculation of failure loads and the application of a suitable factor of safety. Of course, a separate consolidation analysis would have to be made on cohesive soil to guard against detrimental differential settlement.

The above remarks should not be construed to mean that the writer holds plate tests in low regard but only that, perhaps, the above outlined approach offers a sounder solution in that ϕ and c could be accurately determined in a multiplicity of locations very economically. The value of plate tests in obtaining the value of the foundation modulus (k) for use in pavement design is unchallenged; for general home building, plate tests can be incorporated into codes which are compatible with local soil conditions and economy. Also, on very large projects, spot plate testing could serve as a necessary check.

The writer certainly concurs with the author that plate tests are not the place to economize with the cheapest rather than the best. The author is to be congratulated for his thoroughness of detail in describing the test procedure and pitfalls.

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